this Issue:—Experience with Hypochlorite of Lime for Water Purification in Six American Cities.—A Sliding Scale of Electric Power Prices.—Design of Groined Arches for Filter and Reservoir Roofs.—Engineering Material Prices.

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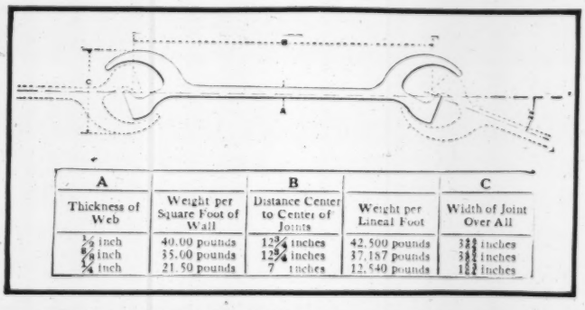

A		B		C
Thickness of Web	Weight per Square Foot of Wall	Distance Center to Center of Joints	Weight per Lineal Foot	Width of Joint Over All
½ inch	40.00 pounds	12¾ inches	42.500 pounds	3⅛ inches
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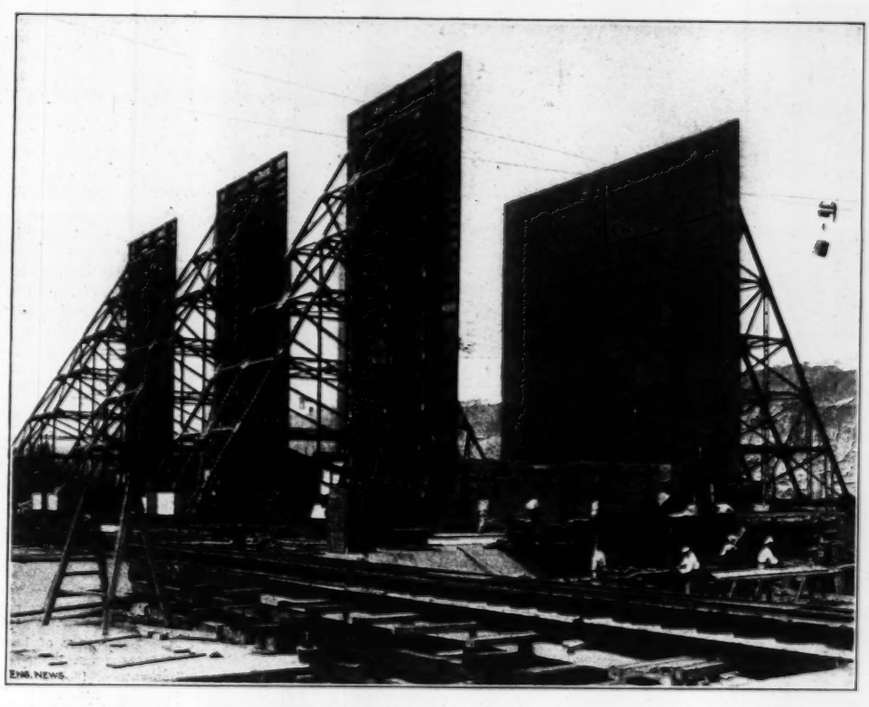
Laying Concrete for the Gatun Locks at Panama.

We reproduce herewith two photographs taken last month, showing the work in progress on the Gatun locks at Panama. The concrete-lock work, of course, involves the construction of very long stretches of high vertical walls. To make these walls with ordinary wooden forms would require an immense amount of falsework and bracing. Portable steel forms have therefore been designed, and several of them are shown in the view on this page. As there seen, the forms against which the face of the lock wall is built are supported on car trucks which travel on lines of rails laid in the lock pit parallel with the axis of the lock. When the form is moved into position it is supported by jacks attached to the towers. There are twelve of these portable face form towers in use at Gatun. Each is 78 ft. high and 36 ft. wide. The lock walls are built in sections 36 ft. long, and grooves are left in the ends of the sections to secure a good bond when the intermediate sections are built.

The cut on the following page shows the concrete work in progress on the foundations of the middle lock of the Gatun flight. Particularly noticeable are the vertical steel rails embedded in the concrete by which the whole lock structure is anchored down to the rock on which it is founded. Up to the end of February, 225,900 cu. yds. of concrete had been laid in the Gatun locks; but about 110,000 cu. yds. of this was laid in the months of January and February. Concrete work is now progressing at Gatun at a rate that requires about 3,300 bbls. of cement daily.



STEEL FORMS FOR THE VERTICAL FACES OF THE LOCK WALLS AT GATUN, PANAMA.

(From a recent photograph by Underwood & Underwood, N. Y.)

NEW ELECTRIC LOCOMOTIVES have been put in service by the Baltimore & Ohio R. R. on the so-called Belt Line in Baltimore, Md. These are of the symmetrical, articulated-truck type in which all tractive and buffing stresses are carried by the truck frame, which is independent of the cab. Each locomotive weighs 90 tons and is rated for a maximum tractive effort of 45,000 lbs. or about 1,750 HP. maximum.

The Mille Lac Lake Survey, Minnesota; Methods and Costs.

By ROBERT FOLLANSBEE,* Assoc. M. Am. Soc. C. E.

By the action of the last session of the Minnesota Legislature, cooperation looking to the investigation of the state's water resources was effected between the State Drainage Commission and the U. S. Geological Survey, whereby the actual work was to be carried on by the latter in accordance with its standard methods. This work comprises not only the study of the flow of the various rivers but also surveys of rivers and lakes for power and storage purposes.

For some years the question of using Mille Lac Lake as a source of water-supply for either St. Paul or Minneapolis has been agitated, but no actual investigation except of the most general nature has ever been undertaken, so far as the writer knows, until recently. Mille Lac Lake, is roughly rectangular in shape, with its greatest diameter about 18 miles in length. It is located principally in Aitkin and Mille Lacs counties, about 85 miles north of the twin cities. As the State Board of Health has certain jurisdiction over the water-supplies of the various cities it was interested in Mille Lac Lake and accordingly cooperation between the State Drainage Commission, the U. S. Geological Survey and the State Board of Health was arranged whereby the Geological Survey was to make a survey of Mille Lac Lake for the purpose of determining the feasibility of using it as a source of municipal water-supply.

Mille Lac Lake already serves as a natural reservoir for water-power purposes on Rum River, its outlet, and thus it is important in that connection also.

The shores of Mille Lac Lake are low and as the distance across the lake varies from 14 to 18 miles it was decided that the increased accuracy gained by having the survey controlled by a triangulation system would not warrant the extra expense, which would have been heavy owing to the necessity of erecting high observation towers. Accordingly the basis of the survey was a closed traverse line 73.2 miles in length around the lake made by transit and stadia. The transit used was a 30-in. Bausch & Lomb instrument and the stadia boards were painted in mass figures to enable them to be read easily at a distance. To insure the stadia boards being held vertical for each sight, rod levels were attached to them. Azimuth was carried by the transit and this was checked by observations on Polaris every ten miles or less. Distances were read forward and back, giving an independent check on each distance. During the first part of the survey the transit was oriented by plunging the telescope when taking the back sight, but this method proved unsatisfactory and in the latter part of the traverse the back sight was taken by keeping the telescope direct, and swinging 180° by vernier from the forward azimuth.

As the survey covered such a large area (somewhat greater than 200 sq. mi.) the traverse was computed by the methods outlined in Wil-

*Old Capitol Building, St. Paul, Minn.

son's "Topographic Surveying," which takes into account the curvature of the earth. This computation gave a closing error of 144 ft. easting, and 106 ft. southing, or 179 ft. in a total length of 386,557 ft.

On the main traverse the party consisted of a transitman, recorder and two rodmen. In addition to keeping notes, the recorder sketched in the shore topography by means of side shots and pacing with the hand level. The object of the shore topography was to show the location of the 5-ft. contour above the low-water lake level. It was not possible to show this location at all points because there were numerous areas of low land lying back of the lake. At such places, magnetic traverses were run to obtain data for locating the 5-ft. contour, if possible. These traverses were made by the method employed by the U. S. Geological Survey in Minnesota on river surveys, and consisted of a line of levels carried by transit with directions determined by magnetic needle and distances by stadia.

SOUNDINGS.—The traverse line just described served not only as a basis for the shore line topography, but also for the hydrographic work. The latter was divided into two parts which were the deep soundings extending across the lake, and the inshore soundings which located the contours of two and five foot depths.

The deep soundings which comprised eleven lines of soundings crossing the lake like lattice work were made by means of a small steamer chartered for the purpose. The beginning and ending of each course was referenced to the nearest traverse station by means of the transit, while between these points the steamer was kept on a straight course by compass and held at as uniform a rate of speed as possible. This speed was about 4 mi. per hr., as it was found impossible to get satisfactory soundings where the depth varied from 20 to 35 ft., when a greater speed was attempted. Soundings were made at intervals of either 3 or 5 minutes by means of a weight attached to a tagged chain. The data just indicated, together with the time of each sounding and the time of commencing and ending each course, were sufficient to locate the soundings with the accuracy needed. A tagged chain was used instead of the customary sounding line to eliminate the correction due to stretching the line. The sounding party, in addition to the regular crew of the steamer, consisted of four men: the chief of party, who laid out the courses for the steamer, and tied in their limits; the recorder, who kept the time as well as recorded the depths; and two leadsmen, who took turns sounding.

The inshore soundings were made by a party of two men with a row boat. This work was done so late in the fall that all the motor boats upon the lake were stored for the winter leaving no other boats available. The party circled the lake locating the point of 2-ft. and 5-ft. depth, at least every mile, by transit and stadia from the nearest traverse station.

SIZE OF PARTY AND COST OF SURVEY.—The party engaged on the Mille Lac survey consisted of four men: a chief of party (who also acted as transitman), a recorder (who also sketched topography), and two rodmen. One of the latter acted as teamster for the party in taking the party to and from work. Fortunately for the efficiency of the survey a road circles the lake, following in most places the shore line. This enabled the party to subsist at hotels and farm houses from ten to fifteen miles apart thus giving not more than seven or eight miles as a maximum drive to reach the work. The party worked eight hours per day, exclusive of time going to and from work. The cost of subsistence per man averaged 87 cts. per day.

The party was in the field 58 days, divided as follows:

Main traverse	23 days.
Secondary traverses and levels	12 "
Deep soundings	3 "
Inshore soundings (2 men)	6 "
Computation	2 "
Traveling to and from St. Paul	4 "
Sundays	8 "
Total	58 days.

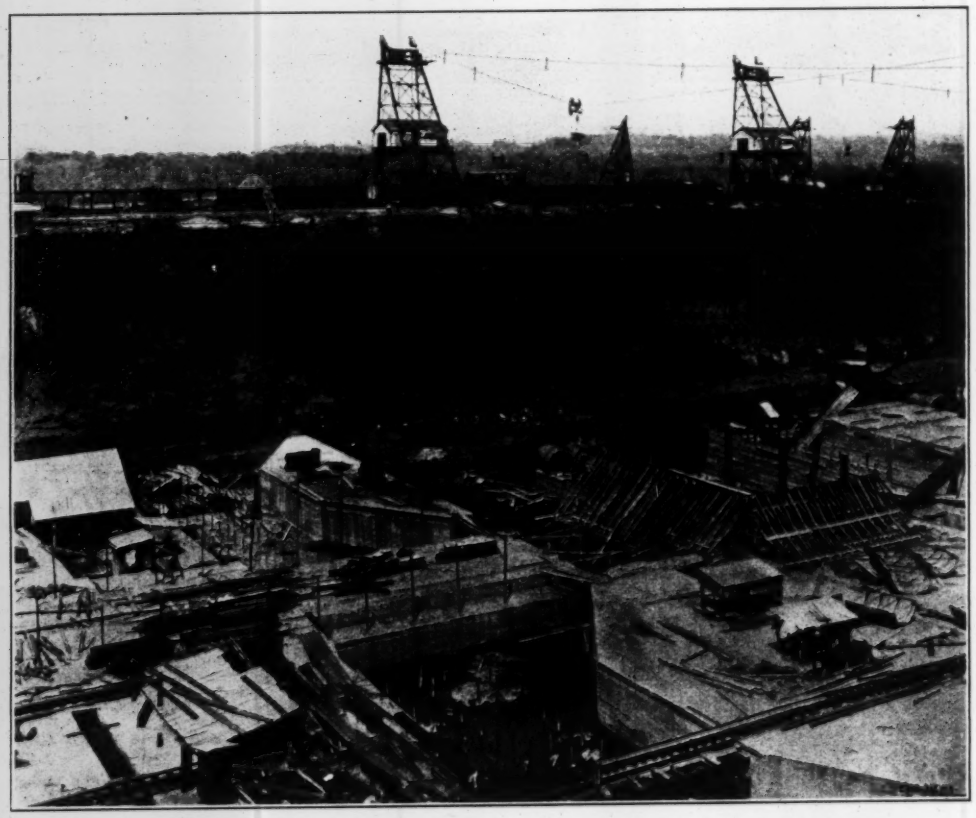
The entire field cost of the survey regardless of supervisory charges was $953.71, divided as follows:

Services	$487.00
Travel*	261.36
Subsistence	187.80
Miscellaneous	17.55
Total	$953.71

*Includes hire of team for greater part of time.

Dividing the entire cost among the different phases of the work gives the following (approximate):

Main traverse	$500	per mile, $6.83
Deep soundings	142	for 392 soundings.
Inshore soundings	52	for 176 soundings.
Magnetic traverses	260	
Total	$954	



CONCRETE WORK IN PROGRESS ON THE MIDDLE LOCK AT GATUN, PANAMA.
(From a recent photograph by Underwood & Underwood, N. Y.)

It is believed that the above figures are considerably lower than that of similar work done elsewhere.

The writer, as district engineer of the U. S. Geological Survey for the Minnesota district was in general charge of the work with Mr. C. E. Adams as chief of party. Many of the foregoing data have been taken from Mr. Adams' report of the survey.

The Use of Sulphate of Alumina and Hypochlorite of Lime in the Storage and Distributing Reservoir of the Nashville Water-Works.

By GEORGE REYER.*

The water-supply of Nashville, Tenn., is pumped from the Cumberland River. At times the river water is turbid and its bacterial content is quite high. A storage and distributing reservoir, built in 1888, effects considerable improvement in the water. This betterment has been increased since May, 1908, by the use of sulphate of alumina, and has been still further increased since August, 1909, by hypochlorite of lime.

The reservoir has a capacity of 51,000,000 gals., or about 3½ days supply. It approximates an ellipse in shape, having axes of 463 and 603 ft. Its water depth is 31 ft. A division wall along the minor axis divides the reservoir into two basins of equal capacity. The gate-house is at one end of this wall and the weir between the two basins is at the other end.

Hourly observations with floats have indicated that the water passes around the curved ends of the reservoir instead of taking the short cut along the division wall.

The sulphate of alumina is admitted to the water as it enters the first basin and the hypochlorite of lime as the water passes over the weir into the second basin. Ordinarily 1 gr. of sulphate of alumina and 0.1-gr. of hypchlorite of lime per gal. of water are used. The sulphate of alumina removes nearly all the suspended matter, so the water is fairly cleared when it passes into the second basin; it also removes quite a percentage of the bacteria. The hypochlorite destroys a large porportion of bacteria, particularly the pathogenic germs.

The following report by Dr. Wm. Litterer, of Nashville, shows the results of a number of bacterial examinations made in January, 1910:

Board of Public Works:

Gentlemen: I herewith submit for your consideration the bacteriological analyses of samples of water collected on Jan. 19, 1910.

The Intake Sample shows a bacterial content at room temperature of 6,800 and at incubator temperature a count of 6,100 per cc. Many acid colonies found on the litmus lactose agar. Of 25 samples examined for B. Coli 92% were positive.

The Reservoir Sample has a bacterial count at room temperature of 110 and at incubator temperature of 80. No acid colonies found on the lactose agar. No Coli were present in 25 samples examined.

The Hydrant Sample shows at room temperature a bacterial count of 150 and at incubator temperature, 100. No acid colonies were present on the litmus lactose agar. No Coli found in 25 samples examined.

The percentage of bacterial elimination from the reservoir at room temperature is 98.4% and at incubator temperature, 98.7%.

The percentage of bacterial removal from the hydrant at room temperature is 97.8%, and at incubator temperature is 98.4%.

Respectfully,

Wm. Litterer, M. D.

Nashville, Tenn., Feb. 1, 1910.

It will be seen that the raw water showed a high number of bacteria and the treated water but few; also that a large percentage of the raw-water samples showed B. coli, while no coli were found in the treated water.

The results of a chemical analysis made a little later, by Dr. W. H. Hollinshead, follow:

Vanderbilt University, Nashville, Feb. 9, 1910.

Nashville Water-Works: I have the honor to report the following result of the analysis of a sample of water taken from faucet at City Hall, Feb. 1, 1910, at 8.3

*Superintendent of Water-Works, Nashville, Tenn.

a. m., by Geo. Reyer. Faucet permitted to run five minutes before taking sample. Container rinsed three times:

(In parts per 1,000,000.)

Sediment	Very small
Odor (heated to 100° F.)	Slightly earthy
Chlorine as chlorides	4.000
Nitrogen as nitrites	Bare trace
Nitrogen as nitrates	0.4000
Nitrogen as free ammonia	0.0318
Nitrogen as albuminoid ammonia	0.0176
Hardness (temporary), equivalent	26.200
Hardness (permanent), to $CaCO^3$	55.000
Hardness (total)	81.200
Total solids (by evaporation)	115.000
Loss in ignition (organic and voltaile)	26.000
Non-volatile (mineral)	89.000
Oxygen consumed	0.835
Alum	None

W. H. Hollinshead.

The turbidity of the raw water is liable to sudden variations, increasing rapidly with a sudden rise of the river, while at other times it is quite clear. Turbidity readings with a platinum wire are made daily. In August, 1909, on beginning the use of hypochlorite of lime, the minimum depth at which the platinum wire could be read when the river was muddy was about 25 mm., maximum depth when the water was fairly clear, was 533 mm. The minimum depth at which the platinum wire could be read in the treated water was 1,397 mm., and the maximum depth was 3,708 mm.

At the same time the bacteria ranged from 6,200 to 1,200 per cc. in the raw water and from 190 to 50 in the treated water. No observations designed to show the effect of sulphate of alumina alone have been made.

The receiving basin, or first half of the reservoir, was cleaned in September, 1909, after having been in use without cleaning for about a year. Approximately 4,500 cu. yds. of sediment were removed, or about 1 cu. yd. per 1,000,000 gals. passed through the reservoir. The other half of the reservoir, termed the clear-water basin, contained very little sediment and was not cleaned.

Definite information regarding the amount of sediment removed by the reservoir before sulphate of alumina was used (that is, by sedimentation alone) is not available; probably it was about one-sixth the present amount.

Dosing Apparatus.

SULPHATE OF ALUMINA.—The apparatus for introducing the sulphate of alumina consists of two wooden tanks, each of about 1,100 gals. capacity. These tanks have a wooden screen about 8 ins. above their bottoms. The alumina is fed to the tanks in regular charges every 15 minutes. A constant stream of water enters the tanks under the screens, and passes off through an overflow into a pipe. This pipe enters an upturned ell 36 ins. in diameter, through which all of the water from the pumps enters the reservoir.

The pump that supplies water to the tanks is driven by an electric motor, and supplies a constant flow of about 40 gals. per min.

The cost of this dosing apparatus was about $500 or $600. The feeding of alumina is done by the watchman, and there is no extra labor required.

HYPOCHLORITE OF LIME.—The hypochlorite of lime is introduced into a tank holding about 1,000 gals. This tank has a vertical shaft in its center and on this shaft are wooden blades to agitate and mix the solution. The bottom of the tank is cone shaped, and at the bottom of the cone is a 1½-in. pipe connected with a small centrifugal pump. The discharge of this centrifugal pump is so arranged that the water can be taken from the bottom of the tank and returned to the tank near the top, thus thoroughly stirring the hypochlorite of lime, and at times the insoluble material can be removed with this centrifugal pump by opening a valve on the discharge that acts as a blow-off.

The power for the pump that supplies the tank, the centrifugal pump and the agitating shaft is obtained from a 2-HP. motor.

The expense of installing this plant was about $400. The expense of operating it is $6 per day.

UNIT COSTS.—The cost of sulphate of alumina is $1.07½ per 100 lbs. and of the hypochlorite of lime is $1.50 per 100 lbs.

We use the ordinary sulphate of alumina. The hypochlorite of lime contains about 36% of chlorine.

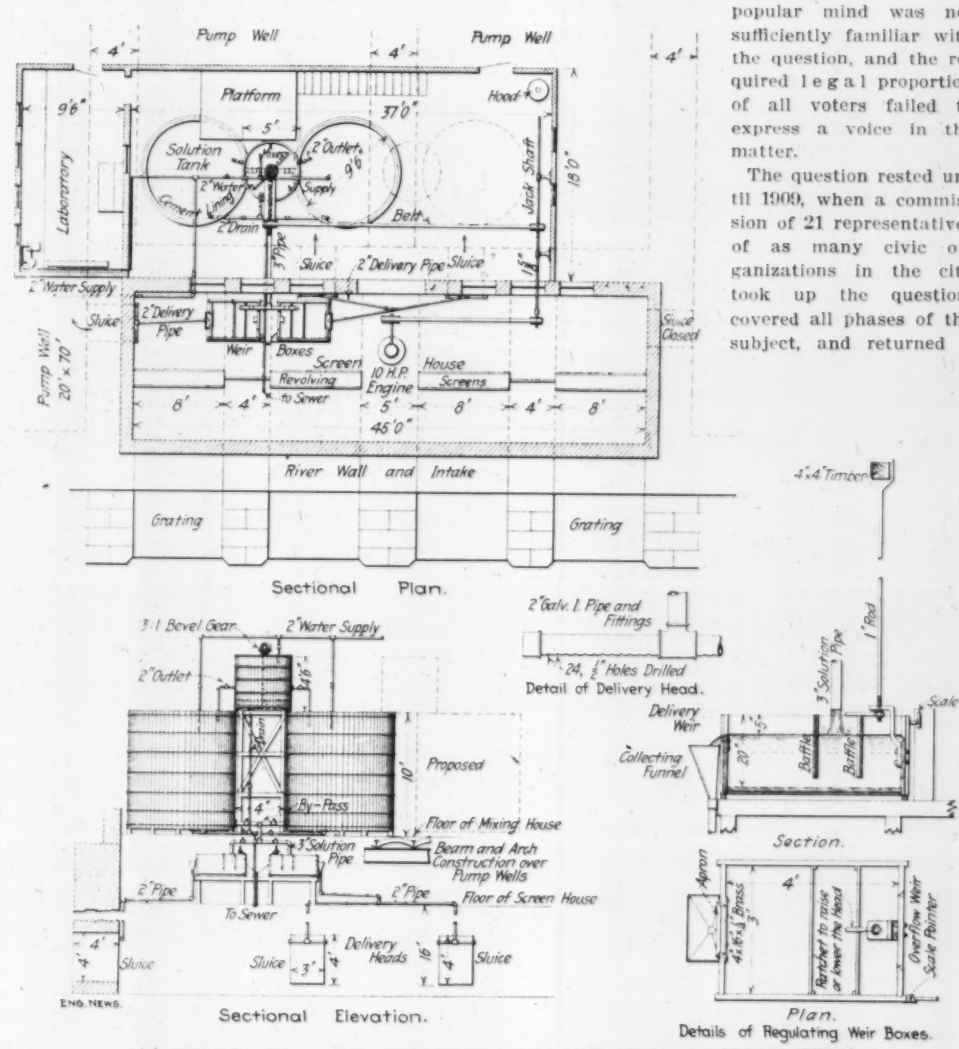
The cost per 1,000,000 gals. of water treated is about $1.75 for sulphate of alumina and $1.05 for cost of hypochlorite of lime, making the combined cost of chemicals $2.80.

The water consumption for the year 1909 averaged about 14,000,000 gals. or some 107 gals. per capita. For the month of January the consumption averaged 12,954,010 gals. and for October it averaged 15,148,000 gals.

A 20,000,000-Gal. Hypochlorite Water-Disinfecting Plant at Minneapolis, Minn.

By J. A. JENSEN.*

The city of Minneapolis has installed a plant to treat its water-supply with hypochlorite of calcium. The initial operation began Feb. 25, and the successful results obtained in the face of a severe outbreak of typhoid promises to retain the treatment until either a new supply is obtained or a purification plant is installed.



HYPOCHLORITE WATER DISINFECTING PLANT AT MINNEAPOLIS, MINN.

The water-supply of Minneapolis is obtained from the Mississippi River at the northern limits of the city and is pumped to a reservoir three miles distant from which it flows by gravity to the distributing system. The reservoir has two rectangular basins, with a total capacity of 97,000,000 gals., and is about 220 ft. above the mean water level of the river. During the past decade the small cities on the upper Mississippi and its tributaries have grown rapidly and four cities now having a total population of 30,000 drain their sewage into the river. The water-supply has deteriorated in desirability and quality and Minneapolis is constantly menaced by sewage pollution.

Active agitation of the water question began in 1904 when a similar outbreak of typhoid† resulted in the permanent closing of two pumping stations which were located in the heart of the city just above St. Anthony Falls and operated by water power. The intakes were in constant danger of pollution from the city itself. A commission was appointed consisting of Messrs. Allen Hazen, of New York, F. W. Cappelen, of Minneapolis, and Andrew Rinker, City Engineer, to investigate and to report to the city council on the question of a new supply or purification. Artesian wells were rejected on account of the unsatisfactory quality of the water. The Lake Superior and Mille Lacs projects were not recommended on account of the prohibitive expense. A report was made favoring slow sand filtration of the present supply as the most suitable and economical process.

The question of bonds was put to a vote at the general election in 1904, but was lost because the popular mind was not sufficiently familiar with the question, and the required legal proportion of all voters failed to express a voice in the matter.

The question rested until 1909, when a commission of 21 representatives of as many civic organizations in the city took up the question, covered all phases of the subject, and returned a report recommending purification of the Mississippi River water. Four of the commission disagreed with the majority and returned a separate report favoring a supply from Mille Lacs, a lake 80 miles distant. This lake is located in a sparsely settled section of former lumbering operations. The water is rather shallow, but is clear and soft and almost free from pollution. It has a surface area of about 200 sq. mi. During the hot months the temperature of the water is quite high and there is a prevalence of algae growth, but at other seasons the water is probably of good quality.

The city council determined to adopt one of the projects and secured Mr. Rudolph Hering, of New York, to determine the relative merits of the two systems. The matter was placed in his hands in November and on March 17, 1910, he reported in favor of continuing the use of the Mississippi River, but advised that it be treated

*Engineer, Water-Works Department, Minneapolis, Minn.

†See Eng. News, March 3 and April 28, 1904.

by mechanical filtration, apparently in conjunction with disinfection by means of hypochlorite of calcium or sodium.

During the early part of the winter the prevalence of typhoid at Brainerd endangered the cities below. At Minneapolis the number of typhoid cases in January began to increase to an alarming extent, and by the middle of February the typhoid death rate was four times the normal for the year. The accompanying table shows the typhoid deaths for the corresponding months in 1909 and 1910:

	1909.	1910.
January	1	11
February	1	31

Investigation showed that 75% of these cases were due to the use of the river water. The matter was taken up with the City Health Department by Mr. Frederic H. Bass, Engineer of the State Board of Health, who proposed the hypochlorite of calcium treatment. The committee on water-works acted at once and the city engineer, Mr. Andrew Rinker, was authorized to construct as expeditiously as possible an emergency plant to treat 20,000,000 gals. per day at the pumping station. This point of application gives the required mixing of the solution in the long force main leading to the reservoir and the latter affords the additional benefit of sedimentation.

The pumping station is located 70 ft. back from a river wall in which the intake gratings are placed parallel to the direction of the current. The water is drawn through a set of forebays and revolving screens from which it passes through sluiceways into rectangular pump wells. These wells are covered with a framing of I-beams, flat brick arches and sod. The sterilizer house and laboratory are built on this covering and adjacent to the screen house so as to give ready access to the water flowing into the wells below. A plan and sections of the plant are shown herewith.

The plant consists of a 400-gal. mixing tank equipped with a vertical shaft and a revolving arm having steel paddles which form a stirring rake. Three baffles are placed on the inside of the tank to prevent the water from swirling as one mass. The shaft has a 3 to 1 bevel gear and a 12-in. pulley on the horizontal shaft. The mixing tank is set on framework above two solution tanks 9.5 ft. in diameter, 10 ft. high, each having a working capacity of 4,800 gals. These tanks are connected with a 3-in. by-pass pipe having a branch running to two regulating weir boxes. Each tank has a water-supply connection and a drain. The weir boxes are 4 ft. long by 3 ft. in width and 20 ins. deep. Each box has a supply weir and an overflow weir and is adjustable by raising or lowering the overflow end by a ratchet on a 1-in. threaded rod by which the adjustable end is suspended. A receiving funnel is placed at each supply weir and forms the collector for a 2-in. delivery pipe leading to the sluiceways. The supply weir is made of a ¼-in. brass plate and is 12 ins. long. As the supply for one pump is drawn through two sluiceways, its delivery pipe is branched to divide the solution as nearly as possible. At the point of delivery the pipe ends in two capped branches, each 18 ins. long, drilled with 24 ½-in. holes. The power is supplied by a 10-HP. engine in the screen house ordinarily made use of in the periodical removal of sand from the pump wells.

The operation of the plant and laboratory is in charge of Dr. J. Frank Corbett, City Bacteriologist, and his assistants. In a series of experiments it was determined that the water would require from ¼ to ½-grain of hypochlorite per gallon. This is higher than the amounts generally used elsewhere, but as the disinfection of the distribution system had to be made as soon as possible, and as the spring floods are at hand, these amounts were necessary to begin with. The operation began with ¼-grain per gallon mixed to form 0.5% solution. At the rate of pumping this requires 8.7 gals. of the solution to be delivered by each weir. A scale was placed at each overflow weir with a pointer at zero at water level. By raising this the weir was calibrated for each division for future use. Both solution tanks were also calibrated for each 0.1-ft. of depth and supplied with indicating gages which, by timing, show the rate of flow when desired. This serves as a check on the weirs. On account of the change in head when the tanks are drawn down the delivery is kept constant by maintaining a uniform head on the weir. This is obtained by keeping the level of the solution always up to the crest of the overflow.

The mixing is done at intervals in about 10-lb. charges. The outlets from the mixing tank are so placed that most of the insoluble portions remain to be blown out at intervals. While one tank of the solution is in service the other is being mixed and tested for available chlorine. Each tank has a capacity for about eight hours of service.

The proportions will be varied when the distributing system has been thoroughly disinfected until the minimum quantity is determined.

Platings are made from the raw water, and also from the treated water at the pumps, inlet and outlet to the reservoir and at the tap for bacterial examination. The following table shows the condition of the water before and the first week after the treatment:

TABLE SHOWING THE PRESENCE OR ABSENCE OF COLON BACILLI BEFORE AND AFTER TREATMENT.

(Treatment began Feb. 25, 1910.)

Date.	Raw river water	Discharge main 10 minutes after treatment.	Reservoir inlet.	Reservoir outlet.	Tap.
Feb. 24.....	Colon	.....	Colon	Colon	Colon
Feb. 25.....	Colon	.....	Colon	Colon	Colon
Feb. 26.....	Colon	0	0	Colon	Colon
Feb. 27.....	Colon	0	0	0	0
Feb. 28.....	Colon	0	0	0	0
Mar. 1.....	Colon	0	0	0	0
Mar. 2.....	Colon	0	0	0	0

The results show the absence of colon at the city hall laboratory tap after the second day, and there will no doubt be a corresponding proportional decrease in typhoid in due time.

The plant began operation eight days after it was authorized. The tanks will be lined with concrete to prevent the action of the solution on the wood. The plans were made and work done under the supervision of Mr. F. H. Bass, of the University of Minnesota, and Mr. J. A. Jensen, Engineer of the Water-Works Department.

The Use of Hypochlorite of Lime to Disinfect the Water-Supply of Montreal, P. Q.

During the fall and early winter of 1909-10 a severe epidemic of typhoid fever occurred at Montreal and near-by municipalities. Epidemics have occurred at this period for a number of years past and during the last epidemic the public became thoroughly aroused.

Purification by means of calcium hypochlorite was inaugurated as a temporary means of improving the quality of the water-supply.

The cause of the recent typhoid outbreak at Montreal is shown by the accompanying morbidity and mortality table, furnished by the Health Officer of Montreal. The population of Montreal is about 400,000.

WEEKLY MORBIDITY AND MORTALITY TABLE FOR TYPHOID FIVER IN MONTREAL FROM OCTOBER, 1909, TO MARCH, 1910.

Month.	Week ending the	Morbidity.	Mortality.
1909.			
October	2	21	5
"	9	53	7
"	16	81	3
"	23	64	6
"	30	36	6
November	6	17	2
"	13	76	8
"	20	38	6
"	27	77	3
December	4	64	8
"	11	104	11
"	18	126	13
"	25	173	10
1910.			
January	1	276	13
"	8	202	26
"	15	208	20
"	22	58	11
"	29	89	8
February	5	45	19
"	12	27	7
"	19	19	4
"	26	19	5
March	5	11	2
Totals		1,884	203

The water-supply of Montreal is pumped from the St. Lawrence River, and has practically no storage before it goes to the consumer. The main part of the city is supplied by a municipal plant, but a portion of Montreal and a number of outlying towns are supplied by the Montreal Water & Power Co. The company is to build a 30,000,000-gal (U. S.) mechanical filtration plant this year. Neither the supply of the company or the city has the benefit of much storage.

A brief description of the two hypochlorite plants follows.

Plant of the Montreal Water & Power Co.

Work was started on the sterilization plant the first week of January, 1910, and the plant was put in operation on Jan. 18.

The building used for the plant is situated over

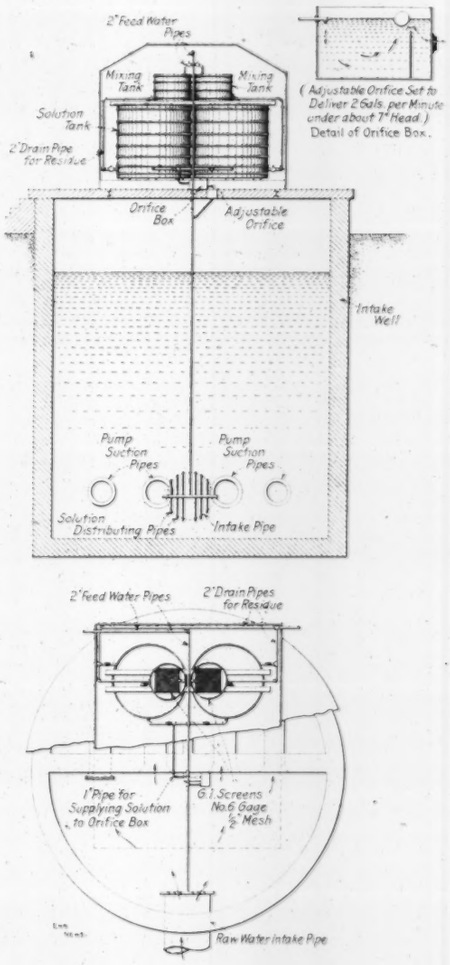

2" Feed Water Pipes
Mixing Tank
Mixing Tank
Solution Tank
2" Drain Pipe for Residue
(Adjustable Orifice Set to Deliver 2 Gals. per Minute under about 7" Head.)
Detail of Orifice Box.
Orifice Box
Adjustable Orifice
Intake Well
Pump Suction Pipes
Pump Suction Pipes
Solution Distributing Pipes
Intake Pipe
2" Feed Water Pipes
2" Drain Pipes for Residue
1" Pipe for Supplying Solution to Orifice Box
G.I. Screens No. 6 Gage ½" Mesh
Raw Water Intake Pipe

Fig. 1. Hypochlorite of Calcium Plant for Treating the Water Supplied by the Montreal Water & Power Co., Montreal, P. Q.

the pump well and is a wooden structure, 20 × 24 ft. in plan and 16 ft. high. The plant (Fig. 1) consists of the following different parts:

Two wooden dissolving tanks, 4 ft. in diameter and 3 ft. deep, placed over the solution tanks. Two solution tanks of wood, 7 ft. 8 ins. in diameter and 7½ ft. deep, having an effective capacity of 2,400 gals.

One orifice box, through which the solution flows, supplied with float-valve which keeps a constant head over the orifice.

Twelve 1-in. perforated pipes connected to the 2-in. feed pipe, which act as distributers of the chemical solution.

Pipes and drains necessary to connect the sup-

ply water to the solution and dissolving tanks, and connections from the tanks to the orifice box and from the orifice box to the branched distributer.

The chemical is weighed and passed through a 1-in. mesh screen into the dissolving tanks, where it is mixed with sufficient water to form a paste. The chemical in this state is well stirred; when well worked up water is turned on and the solution allowed to pass through the overflow of the dissolving tank into the solution tank.

The solution in the solution tanks is stirred hourly by means of a stirrer, made of a piece of perforated boiler plate mounted at right angles to an iron rod. A violent churning action is secured which thoroughly mixes the solution. At the beginning 6 lbs. per 1,000,000 Imp. gals. [0.42 part per 1,000,000 parts or 0.36 per U. S. gal.] was used but later was increased to 9 lbs. Good results were secured with the smaller amount, but because of the rapid variations in the raw water it was thought advisable to use the larger amount.

An attendant is always present at the plant who mixes the solution and puts a fresh tank in service when the first has become exhausted. He also sees that no clogging or stoppage occurs in the feed of the solution.

The cost of installing the plant for sterilizing was $688 for the building, $440 for the apparatus, or a total of $1,128.

The total cost for treating a million gallons is divided as follows:

	Per 1,000,000 gals. Imp.	Per 1,000,000 gals. U. S.
Labor	$0.215	$0.179
Bleaching powder	0.10	0.083
Light and heat	0.015	0.013

The accompanying table gives some details of the use of the disinfectant and the bacterial results obtained.

Mr. J. H. Pitcher is General Manager and Chief Engineer of the Montreal Water & Power Co.

TABLE SHOWING BLEACHING POWDER USED, WATER PUMPED AND BACTERIAL CONTENT OF RAW AND TREATED WATER.

Date.	Bleaching powder, lbs. per million Imp. gals.	Imp. gals. pumped.	Bacteria per cc. Raw water.	Bacteria per cc. Treated water.	Per cent. bacteria removed.
1-31-10	6	16,700,000	1,600	46	97.15
2- 1-10	6	17,300,000	1,500	32	91.2
2- 7-10	6	-7,350,000	240	5	97.92
3- 3-10	9	17,700,000	110	13	88.18
3- 4-10	9	17,800,000	55	4	92.73
3- 5-10	9	17,500,000	29	35	Increase
3- 8-10	9	17,700,000	750	2	97.73

Montreal City Plant.

The city plant was delayed because it was necessary to import the solution tanks from the United States, but the work was finished on Feb. 8, 1910.

The sterilization plant (Fig. 2) is situated near the concrete conduit which brings the supply from the St. Lawrence River to the pumps and is enclosed in a frame building 75 × 28 ft. in plan and 22 ft. high.

The plant closely resembles the one installed by the Montreal Water & Power Co. and consists of the following different parts: Two wooden dissolving tanks 4½ ft. in diameter and 3 ft. deep, situated near the solution tanks. Three solution tanks of wood, 14 ft. in diameter and 10 ft. deep, with an effective capacity of 7,600 Imp. gals. each.

One orifice box with calibrated orifice, the head being controlled by a float-valve.

Pipes and drains necessary to connect with the supply water and the dissolving and solution tanks and with the orifice box and branched distributer.

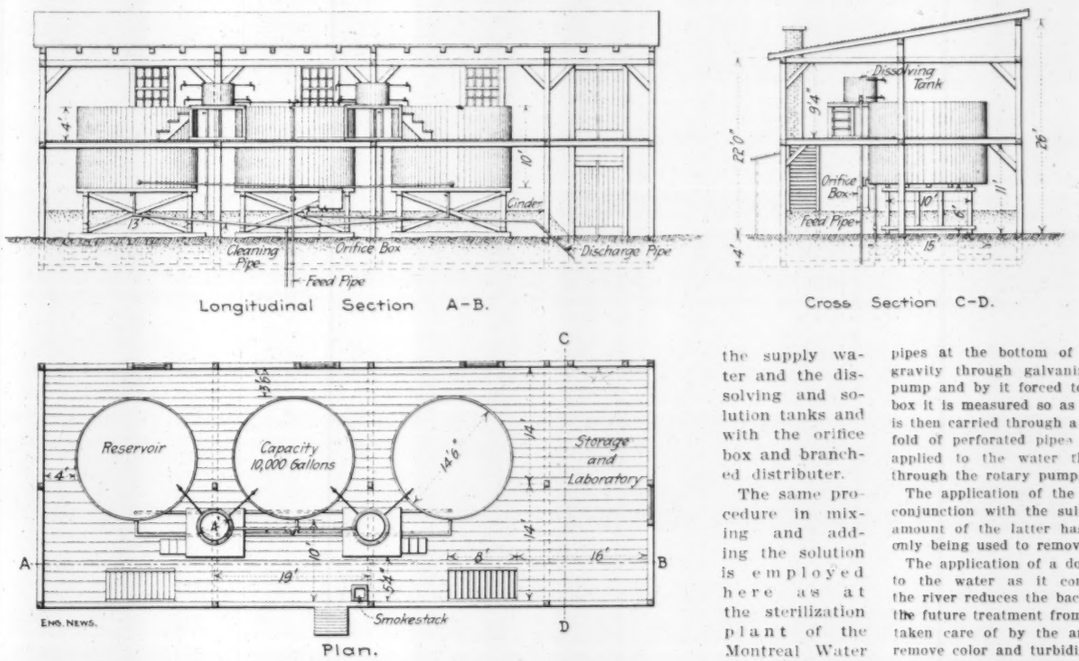


FIG. 2. HYPOCHLORITE OF CALCIUM PLANT, CITY WATER-WORKS OF MONTREAL, P. Q.

The same procedure in mixing and adding the solution is employed here as at the sterilization plant of the Montreal Water & Power Co. At present, the solution is stirred by hand, but it is to be mixed by mechanical means by power furnished from a water motor.

This plant treats 40,000,000 Imp. gals. per day, and is at present operated by two attendants, but this number will be cut in half when the mechanical stirrers are installed.

The cost of the building and plant was $5,770, which included the laying of 400 ft. of water pipe.

The cost for treating a million gallons is divided as follows:

	Per 1,000,000 gals. Imp.	Per 1,000,000 gals. U. S.
Labor	$0.225	$0.188
Bleaching powder	0.100	0.083
Light and heat	0.035	0.029
	$0.36	$0.29

No comparative bacterial results have been obtained as yet, because no provision has been made for securing raw-water samples.

Mr. Geo. Janin is Superintendent and Chief Engineer of the Montreal Water-Works.

The Use of Hypochlorite of Lime in Connection with the Mechanical-Filtration Plant of Harrisburg, Pa.

The Water Department of Harrisburg, Pa., installed not long ago a mechanical-filtration plant which has been considered by many as a model of its kind. The plant treats water taken from the Susquehanna River. Until August, 1909, reliance was placed upon sulphate of alumina as a coagulant, with the addition of soda, when the alkalinity of the water was insufficient. During 1909, up to August, soda was required 47 days. Practice up to the month named was to apply the sulphate of alumina to both the sedimentation and coagulating basin or only to the latter, depending upon the condition of the water. During July, 1909, laboratory experiments were made to test the efficiency of hypochlorite of lime when used in connection with the sulphate of alumina. The hypochlorite was used 8 days in August and 21 days in September, and from September 25 until the end of the year it was used continually. According to the report of the Commissioners of the Water and Lighting Department of Harrisburg for the year 1909, the hypochlorite gave "very gratifying results."

From the report just named we quote an account of the methods employed in the use of hypochlorite, together with a summary of the results which it gave:

The powder is mixed in the tanks provided for soda, a ½% solution being made up. The charge is usually about 100 lbs. and lasts about 24 hours.

It is purchased from the Pennsylvania Salt Manufacturing Co., and lately has been put up in 100-lb. cans. It contains an average of 32% available chlorine, the strength varying a little from the time it is mixed until it is completely used.

The solution is kept to a constant density by agitation by compressed air forced through perforated galvanized pipes at the bottom of the tank and is delivered by gravity through galvanized-iron pipes to the solution pump and by it forced to an orifice box. At the orifice box it is measured so as to give the amount desired and is then carried through a galvanized-iron pipe to a manifold of perforated pipes located in the intake well and applied to the water there immediately before going through the rotary pump.

The application of the hypochlorite is always used in conjunction with the sulphate of alumina, although the amount of the latter has been reduced considerably, it only being used to remove the turbidity and color.

The application of a dose of from .05 to 1-gr. per gal. to the water as it comes into the pump well from the river reduces the bacteria to so small a number that the future treatment from a bacterial standpoint is amply taken care of by the amount of alumina necessary to remove color and turbidity.

From experience gained in the short time it has been used .07-gr. per gal. seems to give results that are perfectly satisfactory. In fact this amount has taken care of water containing as high as 80,000 bacteria per cc.

During the months of October and November, the river water was low in turbidity, color and bacteria, but the proportion of the pathogenic germs to the total number was high on account of the sewage being constant. The average number of bacteria in the river water, the result of the treatment and the coagulant used in these months are as follows:

	Bacteria River.	Bacteria Treated.	Bacteria Filtered.	Grs. per gal. Sulphate of alumina.	Grs. per gal. Hypochlorite.
October	1,633	23	3	.47	.091
November	754	28	5	.47	.054
December	26,804	452	11	.73	.069

Out of 80 1-cc. samples of river water sown for B. Coli, 47 or 58.7% were positive and out of 305 sown; of the filtered water, none were positive.

In December the first bad water in which hypochlorite was used, was experienced. The river had been unusually low for a long period, and the slight rise at that time washed down the several months' accumulation of filth that had collected in the eddies and dams. As the rise was only about two feet the organic matter was not diluted to any great extent. This water, using alumina alone, would have been very hard to handle and would undoubtedly have caused trouble, coming as it did so suddenly and so heavily charged with objectionable matter.

We have added to the summary of results for October and November given above a line showing the corresponding results for December. These results are stated in more detail and for each day in the report, but it does not seem necessary to reproduce the detailed table here. Instead, we will supplement the above-mentioned

figures for December with a few figures taken from the detailed table.

Up to and including Dec. 12, the turbidity of the river water continued uniform at 3 and the color at 2. There was a slight increase in the turbidity, but no change in color on Dec. 13. On Dec. 14 the turbidity suddenly jumped to 160.6, and on the following day it went up to 300. There was then a gradual decline until on Dec. 20 and 21 the turbidity was 40.6 and 10.6, respectively. By Dec. 25 the turbidity had fallen to 5, and by Dec. 30 it was back to 3. The color during this period of high turbidity reached 15 on Dec. 15, and fell to 3 on Dec. 22 and to 2 on Dec. 26.

The bacteria in the river water ranged from 985 to 4,450 during the first 13 days of December. On the day of maximum turbidity for the month (Dec. 16) the total bacterial contents were 85,250, and on Dec. 18, when the turbidity had fallen to 90, the bacterial contents were 86,250. On Dec. 16, when the total bacterial contents were 85,250, the treated water (that is, after having been subjected to the effect of sulphate of alumina, hypochlorite and sedimentation, contained 7,500 bacteria and the filtered water 120). On the same day a total of 1.74 grains of sulphate of alumina were used, of which .7 was added to the water in the sedimentation basin and 1.04 in the coagulating basin. On the same day also the dose of hypochlorite was .059. For the whole month of December the average of .73-grain of sulphate of alumina applied to the water was so distributed that .25 was applied for the sedimentation basin and .48 in the coagulating basin. We quote further from the report as follows:

The available chlorine in the solution during the month averaged slightly over 32%. The amount applied represents .387-part per million of available chlorine or about 9.86 lbs. of powder per million gallons treated.

The action of the hypochlorite is almost instantaneous. The sample of water taken from the cock just beyond the pump shows but a small number of bacteria, yet the water has only passed through the pump and about 15 ft. of pipe.

In order to get a sample of the river water without going to the intake, a small pipe was connected to the pump, and run through the pump well to a point beyond the grid and the controlling valve. A large portion of the samples taken here showed that the bacteria had not been destroyed, but so many of them did show such small numbers that all river samples are now taken out at the intake.

No taste or odor can be noticed from the use of the powder, but at one time when 0.1-gr. was being used a slight taste, that is now thought came from a little algae in the reservoir, caused the application to be reduced to .05-gr. and nothing higher than .075-gr. per gal. has been used since.

From these results it is thought that this material used with judgment will show improved results and will allow all the shortcomings of mechanical filtration, such as air trouble, difficulty with the coagulant in cold weather, difficulty in getting results in water low in turbidity and high in bacteria and other annoyances of the same kind, to be overcome.

The chemical cost can also be reduced considerably and the filters can be run at a faster rate if need be without seriously interfering with the results.

The use of hypochlorite has proved so far to be the best means of eliminating bacteria and has given great relief to the management of the plant.

The services of Prof. James M. Caird are still retained in a consulting capacity and he supervises all work done in the laboratory. This work occupies the attention of the chemist and bacteriologist at all times.

Three samples of the raw and filtered water are tested for turbidity, color and alkalinity and two samples of the river treated and filtered and one of the tap are treated for bacteria. Five one cc. samples each of the filtered and tap are sown each day for B. Coli, and five samples of the river are tested twice a week for the same determination.

A complete chemical analysis of the raw and filtered water is made each week and a mineral analysis occasionally.

While it is unfortunate that some means cannot be devised to get the bacterial contents of the water more quickly than at present, the results of the frequent analysis, even though they only conform or condemn the judgment of the operation, are a great help in the treatment of the water and the operation of the plant.

The long-continued low stage of the river resulted in the hardness of the water becoming noticeable, but the degree of hardness did not reach that of last year. The condition, however, lasted longer than in 1908.

The highest determination of total hardness was 140 parts, 113 parts representing carbonates and 27 parts sulphate hardness.

For the whole year of 1909 the bacterial contents of the river water averaged 5,762, and the filtered water 17, giving a removal of 99.68%. The range in bacterial contents was from 210 in the raw water, as an average for July, with 7 in the filtered water in the same month to 26,804 and 11, respectively, for December. The highest average bacterial content of the filtered water was 123 in January, corresponding to an average of 13,431 for the raw water and a removal of 99.1%. The best month for filtered water was October, when the average bacterial contents of the purified water was 3 as compared with 1,633 for the river water, the removal in this case being 99.71%.

We quote from the report some remarks on the general operation of the plant as follows:

The same general method of operating has also been continued. Air is first given to the filters, the average application for the year being almost four minutes per wash, water for an average of almost five minutes at a vertical height of 9 ins. per minute is then applied, the time from three to six minutes being governed by the turbidity of the water. The filters during the year, however, have been practically filled before putting them in service, thereby lengthening the time off for wash considerably. This, however, could be readily done without any trouble, as the average rate per million gallons per acre per day was only 95,803,243 gals.

The filters are not washed quite clean, as is usually the case, and are not allowed to run to a high loss of head. This taking a little off at each end of a run is done to endeavor to get more uniform results throughout the run. It seems to have been beneficial, but has reduced the average length of runs somewhat, and increased the percentage of wash water from 2.6 to 2.9. The runs, however, are of a little longer duration than last year.

The presence of air in the water during January and the trouble in getting the coagulant to properly react made the results of that month so high that the most careful work was necessary during the balance of the year in order that a record as good as former years could be secured.

The trouble with air was the worst experienced in the history of the plant. The filters would break at a low loss of head and nothing could be done except washing them one after another irrespective of the loss of head in order that the air would not accumulate sufficiently to break through. On some days the filters would be washed at an average loss of head of as low as 3.5 ft. Difficulty was also experienced, at times, in getting any reduction in the number of bacteria by the treatment in the basin.

Mr. George G. Kennedy is Superintendent of the Harrisburg Water Works; Mr. Charles E. Clifton is Resident Chemist and Bacteriologist at the filter plant, and Prof. James M. Caird, of Troy, N. Y., is Consulting Chemist and Bacteriologist.

In response to an inquiry, Mr. Kennedy has written us that the tanks designed for adding soda to the water have been used for adding the hypochlorite. He also says:

The continued use of the hypochlorite has further proven the benefit derived from it. The operations of our plant during the months of January and February—ordinarily the worst months of the year—has been made much easier and the results bacterially and in the quantity of chemicals used are remarkable. The bacteria in the river water averaged 23,117 per cc. and in the filtered water only 6 in January, while in February the river water contained 9,015 and the filtered 3 bacteria per cc., both remarkable averages when it is considered that these months are the ones when all the shortcomings of mechanical filters are present.

Hypochlorite of Lime as an Adjunct to Mechanical Water Filtration at Quincy, Ill.*

By W. R. GELSTON.†

About two years ago, the reports of analyses of the Quincy water, as made by the State Water Survey, began to indicate a decline in the bacterial efficiency of our mechanical filtration plant. Prof. E. W. Bartow, Director of the Survey, suggested the use of bleaching powder as a possible remedy, and offered to furnish a man to instruct us in the use of it and to make bacterial analyses at our pumping station. As an additional inducement Dr. Bartow agreed to leave his man with us long enough to put a permanent bacteriological laboratory in operation and teach us to make our own counts. We gladly accepted the offer, and Mr. G. A. Van Brunt was sent to us March 22, 1909. Our first satisfactory counts were made on samples plated April 6, and since that date we have operated the plant during several periods of time without bleaching powder, and have tried various amounts of the bleach during periods of sufficient length to give fairly reliable results.

We have not, at any time, by the use of bleaching powder, attempted to reduce the amounts of sulphate of iron and lime ordinarily used, because there are certain defects in the construction and arrangement of our plant which prevent a high efficiency even when the condition of the river water is most favorable. Our use of bleaching power was, therefore, entirely for the purpose of increasing the efficiency of the plant.

The original apparatus used for applying the bleaching powder consisted of an oil barrel with one head removed and a short piece of ½-in. pipe inserted near the bottom. The barrel was filled with water, and the required amount of bleaching powder for one or two hours' run was dissolved in it and the rate of flow regulated by a valve in the end of the pipe. Owing to the rapidly changing head of water in the barrel, this valve required much attention. Fortunately, the bleaching powder has a way of its own of removing unsatisfactory apparatus and the barrel soon went to the scrap pile. To take its place, we put in a shallow wooden tank of about 100 gals. capacity when filled to a depth of 1 ft. This proved much more satisfactory. The bleaching powder solution was introduced into the settled water at the point of overflow from the basin and, as the water flows through about 100 ft. of 20-in. pipe before reaching the filters, the solution was quite thoroughly mixed with it.

During the month of April, the work was under Mr. Van Brunt's supervision and samples were plated each forenoon and afternoon when the plant was in operation. After Mr. Van Brunt left us, one sample was usually plated each day from the river, and from the effluent from the settling basin and one from a filtered-water tap on the water chamber of the high duty pump. Occasionally samples were plated from individual filters and from the influent to the settling basin.

For the purpose of showing the effect of using bleaching powder, the following tabulated statement has been prepared, in which will be found, for each period of operation with different amounts of bleach, the average turbidity of the river water; the average alkalinity of the river and tap waters; the average stage of the river; the amount of bleach used in grains per gallon; the average number of colonies of bacteria per c.c. in the river and tap waters, and the average per cent. of removal; also the number of positive and negative tests for gas formers in the river and tap waters. Nearly all tests for gas formers in the river water were made on 0.5 c.c. samples and for the tap water, 1 c.c. samples were used.

During the first period of operation, from April 6 to April 13, no bleach was used and the regular operation of the plant in no way disturbed. April 14, Mr. Van Brunt began using bleaching powder at the rate of 0.25-

*Abstract of a paper read at the annual meeting of the Illinois Water Supply Association, held at the University of Illinois, March 8 and 9.

†Superintendent, Citizens Water Co., Quincy, Ill.

TESTS OF WATER AT QUINCY, ILL., TO DETERMINE EFFICIENCY OF TREATMENT WITH HYPOCHLORITE OF LIME.

Date from to	Average turbidity of river.	Average alkalinity		Average stage of river.	Average Bacterial Contents. Hypochlorite, gr. per gal.	Per c. c.		Per cent. removal.	Gas formers.	
		River.	Tap.			River.	Tap.		River.	Tap.
April 6 to 13, 1909	1,230	120	73	10.0	0.0	8,836	276	96.9	8+	4+ 4−
April 14 to 27, 1909	557	102	65	13.6	0.14 to 0.25	5,423	15	99.7	13+ 2−	6+ 17−
April 28 to May 3, 1909	257	108	63	14.7	0.0	2,193	493	77.5	5+	4+ 4−
May 4 to June 9, 1909	362	115	71	12.9	0.14	3,986	154	96.1	25+	14+ 16−
June 11 to 23, 1909	578	119	76	13.2	0.0	63,443	2,038	96.8	9+ 1−	9+ 1−
June 24 to 25, 1909	180	128	75	10.5	0.10	2,030	36	98.2	2+	2+
June 26 to July 2, 1909	567	125	83	11.4	0.0	7,870	1,265	83.8	4+	5+ 2−
July 6 to 16, 1909	498	132	93	13.1	0.07	22,790	853	96.3	9+	9+
July 17 to Aug. 5, 1909	150	158	98	6.5	0.10	6,283	265	95.8	9+	8+ 1−
Aug. 6 to 23, 1909	138	153	98	4.7	0.0	3,437	942	72.6	5+ 1−	6−
Aug. 24 to 27, 1909	262	123	79	5.4	0.07	6,127	139	97.7	4+	1+ 3−
Aug. 28 to Oct. 9, 1909	165	140	91	3.9	0.0	6,098	96	98.4	22+	18+ 4−
Oct. 11 to 12, 1909	90	144	86	2.8	0.035	2,695	22	99.2	2+	2−
Oct. 13 to 28, 1909	94	151	96	2.8	0.07	3,161	64	98.0	8+	2+ 5−
Oct. 30 to Nov. 8, 1909	94	149	90	3.0	0.10	4,337	72	98.3	3+ 1−	4−

grs. per gal. of water treated, and this amount was continued until the morning of April 17, when complaints of a medicinal flavor in the water, began to come in. The amount of bleach was at once reduced and until April 27 various quantities from 0.25 down to 0.14-grs. per gal. were used, and still we received complaints about the water. We therefore did not use bleach from April 28 until May 3, and a remarkable loss of efficiency for the plant will be noticed in the statement, the average percentage of removal for this period being 77.5 as compared with 99.7 for the period while we were using from 0.14 to 0.25 grs. and 96.9 for the period before any bleach was used at all.

It was very unfortunate that our first attempt to use bleaching powder resulted in an overdose, because this experience developed an abnormal sense of taste and smell in some of our consumers and we were constantly in receipt of complaints about the water for several months. Some of these complaints were just, but a large majority of the complainants were unable to produce any bad water when called upon to furnish samples of it. I do not believe, however, that there was ever any odor or taste due to bleaching powder when we used less than 0.14-gr. per gal. Nevertheless, this first experience had an intimidating effect upon the operating officials of our plant and we discontinued the use of the bleach at the first sign of an approaching storm, thereafter.

A study of the tabulated statement will show that, between April 6, 1909, and Nov. 8, 1909, the plant was operated during six different periods of time without bleaching powder, with a maximum average removal of bacteria of 98.4% and a minimum of 72.6%, the average for the six periods being 87.7. Within the same limits of time, there were nine periods of operation with different amounts of bleach, from 0.035 to 0.25-gr. per gal. with a maximum average removal of 99.7% and a minimum of 95.8%, the average for the nine periods being 97.7% or 10 points higher than the average for the six periods without the bleach.

A comparison of the results of tests for gas formers will show that 96.2% of all samples of river water indicated the presence of gas formers; 65.6% of all samples of filtered water tested, when we were not using bleaching powder, gave positive results and 46.7% of all samples of filtered water tested when we were using bleaching powder gave positive results. In our report on gas formers, we have not used the term "doubtful." If there was any gas in the closed arm of the Smith tube, the test was marked positive.

Experiments with Hypochlorite of Lime as a Water Disinfectant at Hartford, Conn.

By ERMON M. PECK.*

Experiments with hypochlorite of lime for purification of the water of the Connecticut River were conducted recently by the Board of Water Commissioners, at Hartford, Conn.

The apparatus for the experiments was designed by and erected under the supervision of Dr. John L. Leal, the well-known sanitary expert of Paterson, N. J. Dr. Leal had, as his assistant making the tests, Mr. Harold C. Stevens, who was especially released for this purpose by Mr. J. Waldo Smith, M. Am. Soc. C. E., of the Board of Water Supply, of New York City, in whose Department Mr. Stevens is employed. The work was under the direct charge of the writer.

Entirely independent of Dr. Leal, Prof. James A. Newlands, of the Connecticut State Board of Health, was employed as an expert to supervise a set of tests made with the same apparatus, his assistant on the ground being Mr. A. F. Stevenson, a graduate of the Massachusetts Institute of Technology. The apparatus for mixing and introducing the calcium hypochlorite to the raw water, consisted of a 75-gal. mixing tank, a 1,000-gal. solution tank, and a 100-gal. orifice tank. All tanks were of wood and besides being properly pipe connected with each other, were fitted with blow-offs for cleaning.

Raw water for the experiments was supplied by an ancient pumping plant which formerly supplied the city in the less-polluted days of the Connecticut River but now for many years unused. This plant consists of four vertical bucket pumps geared to an old-fashioned walking-beam engine. Water is conducted to these pumps through two 20-in. suction pipes. The solution flowed from the orifice tank through a ½-in. brass pipe, branching to the two intake pipes into the bottoms of which they were tapped.

From the pumps the treated water flowed through about a mile of 16-in. pipe and was wasted into a sewer through an open blow-off, after being measured by a 6-in. Crest meter, the registration of which was read by an observer every 15 minutes.

Samples of water for testing were taken at four points along the force main. These stations were numbered 1, 2, 3, and 4 and were situated respectively 700 ft., 2,175 ft., 3,115 ft. and 4,600 ft. from the point at which the raw water was treated.

A vacant room in the old pumping station was fitted up for a bacteriological laboratory, wherein Messrs. Stevens and Stevenson made the tests. Experiments were begun Dec. 19, 1909, and continued until Jan. 1, 1910.

Table I., from the report of Professor Newlands, shows the bacterial changes at the different stations with varying amounts of hypochlorite. It likewise shows very clearly the effect of the element of time, when very small amounts of chemical are used.

Table II., also from Professor Newlands' report, shows the percentage removal of bacteria throughout the tests, as calculated from the results obtained at Station No. 1, and the presence or absence of B. Coli, in the treated water.

Commenting upon Table II, Prof. Newlands says:

> It will be seen by these results that when one part per million, or more, of available chlorine was used, the removal of bacteria was always greater than 99.5%, and the Colon bacillus was not found in the treated water.

The object of these experiments was somewhat unusual. Hartford's water-supply is by gravity from surface gathering grounds. The safe capacity of the present works is not far from 7,500,000 gals. per day and at present the amount of water drawn from the reservoirs very nearly equals that amount. During the past two years, in spite of the fact that the rainfall has been very little below the average, the reservoirs became depleted to such an extent that the water-works' officials feared a water famine. For many years it has been the custom in the emergency resulting from a water famine to supply a portion of the city with unpurified Connecticut River water.

Recently when the water in the reservoirs began to get low and the local board of health heard that there was danger of resorting to this much-polluted stream, it promptly took action condemning its use and presented the board of water commissioners with a memorial to this effect.

Since the river was practically the only alternative and the danger of a famine was increasing daily, the experiments were conducted with a view to overcoming the objection of the health board and affording temporary relief should it become necessary. In this connection it may be said that the results attained were much better than the water-works officials anticipated.

*Engineer Board of Water Commissioners, Hartford, Conn.

TABLE I.—BACTERIAL ANALYSES OF RAW CONNECTICUT RIVER WATER AND WATER TREATED WITH HYPOCHLORITE OF LIME, HARTFORD, CONN.

Sample taken Dec., 10.	Available chlorine, p. p. m.	Bacteria per cubic centimeter in Raw water Station No. 0		Treated water Station No. 1*		Station No. 2*		Station No. 3*		Station No. 4*	
		20°	37°	20°	37°	20°	37°	20°	37°	20°	37°
19 p. m.	1.62	.....	...	0	23	1	48	3	...	3	26
20 p. m.	2.06	36,000	...	20	26	8	13	...	...	12	22
21 a. m.	1.77	24,600	620	19	30	16	38	...	...	15	26
21 p. m.	1.48	14,000	1,100	19	37	23	54	...	...	23	35
22 a. m.	1.84	19,200	620	13	33	19	48	...	...	20	56
22 p. m.	1.58	10,200	970	14	19	11	17	...	...	15	14
23 p. m.	.51	64,000	310	4,800	18	3,800	31	...	...	140	21
24 a. m.	.53	50,000	660	1,020	28	1,080	38	...	...	100	48
24 p. m.	.56	24,000	670	180	39	320	45	...	...	225	...
27 a. m.	.72	37,600	170	3	3	3	13	...	...	4	4
27 p. m.	1.14	17,200	80	10	25	22	19	...	...	19	15
28 a. m.†	1.29	9,300	310	44	14	944	52	205	14	1,195	50
28 p. m.	1.27	21,800	200	6	16	26	14	8	3	3	10
29 a. m.	.93	29,000	440	21	23	25	32	29	34	35	40
29 p. m.	.95	17,200	270	40	37	30	31	34	22	18	22
30 a. m.	.79	16,000	140	7	27	12	5	...	...	7	13
30 p. m.	.78	15,600	270	7	18	5	14	...	...	12	20
31 a. m.	.80	8,100	210	12	4	6	12	...	...	9	15
31 p. m.	.85	‡	140	‡	7	‡	11	...	...	‡	7

*Station No. 1 was 700 ft. distant from Station No. 0; Station 2 was 2,175 ft.; Station 3 was 3,115 ft.; Station 4 was 4,600 ft.

†Garden Street Reservoir had been connected with the main and reservoir pressure forced water back through the pump.

‡Room temperature tests accidentally spoiled.

TABLE II.—PERCENTAGE REDUCTION OF BACTERIA IN CONNECTICUT RIVER WATER AT HARTFORD, CONN., BY DISINFECTION WITH HYPOCHLORITE OF LIME.

Sample taken Dec.	Available chlorine, p. p. m.	Number of bacteria per c.c. Raw water, Station No. 0.		Treated water, Station No. 1.		Per cent. removal.	B. Coli present in 1 c.c. Station No. 0.	1 c.c. Station No. 1.	5 c.c. Station No. 2.	5 c.c. Station No. 4.
		20°	37°	20°	37°					
19 p. m.	1.62	.....	...	0	23	100.	...	...	...	...
20 p. m.	2.06	36,000	...	20	26	99.94	...	...	...	...
21 a. m.	1.77	24,600	620	19	30	99.92	...	...	...	...
21 p. m.	1.48	14,000	1,100	19	37	99.86	...	...	...	...
22 a. m.	1.84	19,200	620	13	33	99.93	+	—	—	—
22 p. m.	1.58	10,200	970	14	19	99.86	+	—	—	—
23 p. m.	.51	64,000	310	4,800	18	92.50	•	•	•	•
24 a. m.	.53	50,000	660	1,020	28	97.96	+	+	+	+
24 p. m.	.56	24,000	670	180	39	94.25	+	—	+	—
27 a. m.	.72	37,600	170	3	3	99.99	+	—	—	—
27 p. m.	1.14	17,200	80	10	25	99.95	+	—	?	—
28 a. m.	1.29	9,300	310	44	14	99.52	+	—	—	—
28 p. m.	1.27	21,800	200	6	16	99.97	+	—	—	—
29 a. m.	.93	29,000	440	21	23	99.92	+	—	—	+
29 p. m.	.95	17,200	270	40	37	99.76	+	—	+	—
30 a. m.	.79	16,000	140	7	27	99.95	+	+	+	+
30 p. m.	.78	15,600	270	7	18	99.95	+	+	+	+
31 a. m.	.80	8,100	210	12	4	99.86	+	—	—	—
31 p. m.	.85	†	140	†	7	.....	+	—	—	—

*Did not give a definite test for B. Coli.

†Room temperature tests were accidentally spoiled.

GOVERNMENT OPERATED TELEPHONE LINES in Manitoba comprise 5,180 miles of long distance circuits, 88 exchanges and 145 call offices. The second annual report of the Government telephone commissioners shows total receipts for 1909 amounting to $754,000 and expenditures for operation and maintenance of $482,211. There are now $25,300 subscribers, of which 7,000 are farmers living in rural districts, the charge to whom was reduced in April, 1909, from $24 to $20 per year. The total capitalization of the telephone system is $5,102,977. Hon. Robert Rogers of Winnipeg is the Minister of Telephones and Telegraphs of Manitoba.

ANOTHER SHIP MODEL TANK is being built in England. Mr. A. F. Yarrow donated the money for such a tank to be run in conjunction with the National Physical Laboratory, with the advice of a committee of the Institution of Naval Architects. The tank is now in process of construction. It is located at Teddington. The width at top is 30 ft., the depth 13 ft. at center on a length of 493 ft., and the total length 555 ft. The enclosing building is 42 ft. wide inside, having gangways on either side 3 ft. 6 ins. below the top of tank walls. A sawtooth roof covers the building. There is a smaller auxiliary tank, 5 ft. wide by 58 ft. long, with 42-in. depth of water; this is fitted with pumps for creating a flow through it. A preliminary report on this undertaking, with drawings, was rendered to the Institution of Naval Architects by R. T. Glazebrook, Director of the National Physical Laboratory, on March 16, and this is reprinted in "Engineering" of March 18.

An Equitable Sliding Scale for Rates of Electric Power.

By J. B. GORDON,* Assoc. M. Am. Soc. C. E.

The purpose of this article is to develop formulas by which the rates to be charged a consumer of electric power can be ascertained, by substituting his conditions of operation—such as rated capacity of plant and the load factor, in the formulas in order to determine his rates according to a fixed law rather than arbitrarily. Also by plotting curves based on these formulas, the electric company will be enabled to determine, by inspection, the approximate charges for various conditions. This method was used in preparing rate sheets for the sale of power

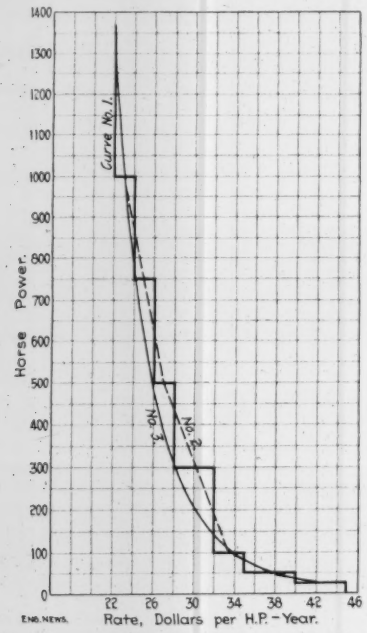


Fig. 1. Diagrammatic Comparison of Arbitrary and Equitable Scale Rates.

Curve No. 1. Showing rates under prevailing practice.
Curve No. 2. Showing mean values of present arbitrary rates.
Curve No. 3. Showing rates resulting from Formula "A."

from a proposed hydro-electric development. As the accepted custom in listing a scale of rates has been modified, the writer believes the work will prove of interest to others.

It should be borne in mind that this discussion does not touch upon questions of charges equitable to the company as well as to the consumer or the increase of dividends for corresponding general reduction of rates,† or on dividing scales.

The principal object in developing these formulas is that sudden changes in charges for electricity (according to the prevailing practice in the section of the country in which this method is to be used) may be eliminated. It seemed that, without diverging much from those rates established by our predecessors, a fixed law could be laid down which would overcome some of the objectionable features and which at the same time would be more comprehensive.

Relation Between Rates and Load.

For instance, assuming a ten-hour day, or 35% load factor, and following the old method of charging, the rates were as follows:

Up	to	25 HP.	$45.00	per HP.-year.
25 HP.	"	50 "	40.00	" "
50 "	"	100 "	35.00	" "
100 "	"	300 "	32.00	" "
300 "	"	500 "	28.00	" "
500 "	"	750 "	26.00	" "
750 "	"	1,000 "	24.00	" "
1,000 "	upwards		22.00	" "

These charges do not seem equitably adjusted, as a consumer with a plant of 950 HP., rated capacity, must pay $24 per horsepower per year, while another consumer with a plant of 1,050 HP. capacity need pay only $22. The minimum charge per horsepower-year was to be kept at $22, and the company decided to make no contracts for less than 25 HP.

From the above table of rates the mean charges are seen to be as follows:

25	HP.	$42.50	per HP.-year.
50	"	37.50	" "
100	"	33.50	" "
300	"	30.00	" "
500	"	27.00	" "
750	"	25.00	" "
1,000	"	23.00	" "

These mean rates were plotted (see Fig. 1), employing the HP. as abscissa and the rates as ordinates. With these values it was seen that no smooth curve could be drawn through all points. As the general shape of the line connecting these points was that of a parabola, an investigation was made to determine whether such a curve could be drawn which would even approximately fit the conditions. On account of the sudden change in direction at about 100 HP., this was found to be impossible.

It then became apparent that a formula must be built up and investigations were made to determine what, if any, relations existed between horsepower and rate, in the existing charges. In making these investigations as to existing relations, the limiting cases only were considered, that is 25 HP and 1,000 HP.

From inspection it is seen that the rate varies inversely as some function of the horsepower, and to determine this function a tabulation was made of HP., $(HP.)^2$, $(HP.)^3$, $\sqrt{HP.}$, $\sqrt[3]{HP.^2}$, $\frac{1}{HP.}$, etc., while opposite these were tabulated \$, \$², $\sqrt{\$}$, etc.

It was seen that

$$\frac{\sqrt[3]{1000\ HP.}}{\sqrt[3]{25 HP.}} = 3.42$$

while that

$$\frac{(\$42.50)^2}{(\$23.00)^2} = 3.41$$

or that the cube root of the horsepower varies approximately inversely as some function of the square of the rate, i. e.,

$$\text{Rate} = \sqrt{\frac{K}{\sqrt[3]{HP.}}}$$

Solving in the above two cases in order to determine the value of "K" it is found that when HP. = 25 and Rate = \$42.50, $K = 5{,}274$; and when HP. = 1,000 and Rate = \$23.00, $K = 5{,}290$.

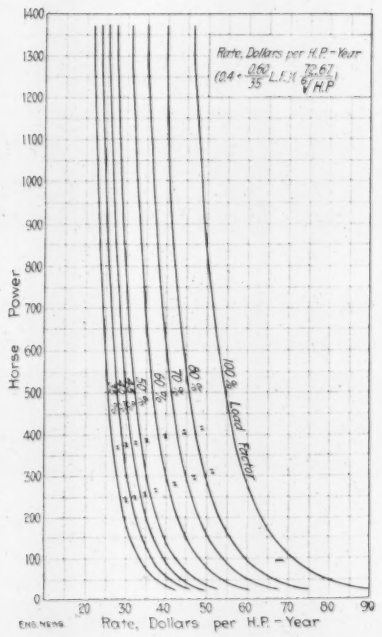


Fig. 2. Curves of Equitable Flat Rates for Electric Power, Computed with Formula "B."

The value of K is taken as the mean of the above two, or as 5,282, and the formula becomes

$$\text{Rate} = \sqrt{\frac{5282}{\sqrt[3]{HP.}}} = \frac{72.68}{\sqrt[6]{HP.}} \quad \ldots\ldots\ldots (A)$$

Substituting in the above formula, the various values for horsepower, the corresponding rates are as follows:

For	25	HP.	$42.50	per HP.-year.
"	50	"	37.86	" "
"	100	"	33.73	" "
"	300	"	28.09	" "
"	500	"	25.80	" "
"	750	"	24.11	" "
"	1,000	"	22.98	" "

Comparing these new rates with the original (Fig. 1) it is seen that there is a considerable difference in some instances. However, this difference was foreseen from the fact that the original rates would not plot on a smooth curve. Moreover, it demonstrates that the old sliding scale of rates was not equitably adjusted.

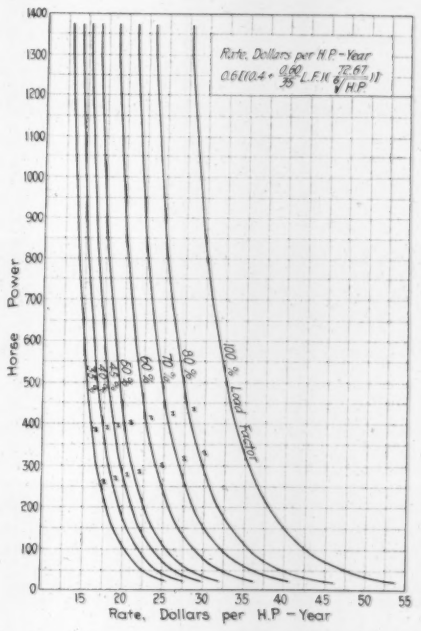


Fig. 3. Curves of Investment and Maintenance Charges, Computed with Formula "C."

Relation Between Rates and Load Factor.

The discussion, up to this point, applies to rates to be charged when the load factor is 35%. This 35% load is equivalent to an operation of ten hours a day for 306 days in the year. Working backwards in formula "A" it is found that the minimum rate of \$22 applies to 1,300 HP. This was fixed as the maximum horsepower for which the formula would apply. Therefore the formulas used in this discussion hold only between the limits of 25 and 1,300 HP. For any consumer desiring more than 1,300 HP., his rate, regardless of load factor, will be the same as for 1,300 HP.

The usual method of giving the consumer a benefit of reduced rates for any increase of load factor over 35% was to be adhered to in this case. But as the rates of this particular case are intended to apply to textile industries only, the company decided to supply no power under this agreement upon a basis of less than a 35% load factor. It therefore becomes necessary to modify formula "A" so that it will apply to larger load factors. It was decided that upon increasing the load factor from 35 to 70%, that is, by a 100% increase, the equivalent flat rate should be increased but 60%. Thus for every increase of 1% in the load factor the rate will be increased $\frac{0.60}{35}$ "A." But as this modification applies only to an increase of load factor over 35 it is necessary to insert an additional factor, making the expression read:

$$\frac{0.60}{35}\text{"A"}\ (\text{Load Factor} - 35).$$

*Gordon and Conard, 1410 H St., N. W. Washington, D. C.

†Such a basis of balancing decrease of rates and increase of dividends was shown in an article by Mr. W. D. Marks, in Engineering News, July-Dec., 1909, p. 231.—Ed.]

With a load factor of 35 this term becomes zero.

The complete formula expressing the equivalent flat rate to be charged per horsepower year becomes, therefore,

$$\text{Rate} = \text{“}A\text{”} + \frac{0.60}{35}\text{“}A\text{”}\,(L.\ F. - 35)$$

$$= \text{“}A\text{”}\left(1 + \frac{0.60}{35}[L.F. - 35]\right)$$

$$= \left(0.4 + \frac{0.60}{35}L.\ F.\right)\text{“}A\text{”}$$

$$= \left(0.4 + \frac{0.60}{35}L.\ F.\right)\left(\frac{72.68}{\sqrt[5]{\text{HP.}}}\right) \quad \ldots\ldots(B)$$

From this it is seen that with a load factor of 35 the expression "B" reduces to "A."

This formula "B" now gives the equivalent flat rate to be charged for power; but in the agreement to be entered into by both parties there is the following clause:

> In consideration of the Company making and maintaining the necessary investment to supply the power, the Consumer agrees to pay to the Company an investment and maintenance charge of ($——) per electrical horsepower, based upon the aggregate rated capacity, whether any power is used or not.

The fixed charge in this case, it was decided, should be 60% of the total equivalent flat rate and consequently can be expressed as follows:

$$\text{Fixed charge} = 0.6\ \text{“}B\text{”}$$

$$= 0.6\left(0.4 + \frac{0.60}{35}L.\ F.\right)\frac{72.68}{\sqrt[5]{\text{HP.}}} \quad \ldots\ldots(C)$$

Also the agreement that provides that

> In addition to the aforesaid investment and maintenance charge, the Consumer shall pay —— mills per kilowatt-hour for all power used.

This meter rate, based on the remaining 40% of the total and so modified as to read in mills per kilowatt hour, reduces to the following expression:

$$\text{Rate per K W hr.} = \frac{400}{87.6\ L.\ F. \times 0.746}\text{“}B\text{”}$$

$$= \frac{400}{65.35\ L.\ F.}\text{“}B\text{”}$$

$$= \frac{400}{65.35\ L.F.}\left[\left(0.4 + \frac{0.60}{35}L.\ F.\right)\left(\frac{72.68}{\sqrt[5]{\text{HP.}}}\right)\right] \ldots(D)$$

The above rates apply only to "Class A" or "primary" power.

For the benefit of the company's agents, tables

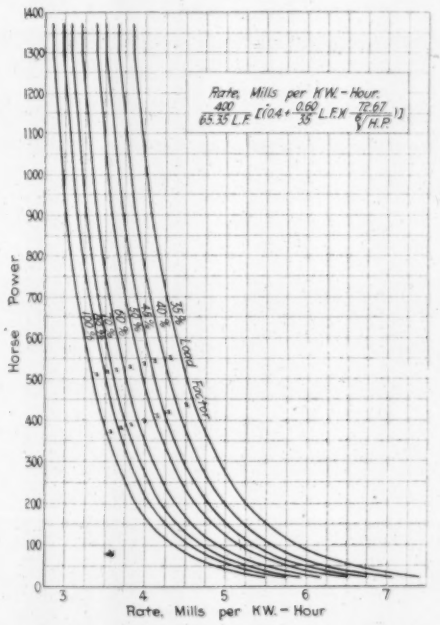


Fig. 4. Curves of Meter Rates, Based on Formula "D."

and curves, based on these formulas, were prepared. By consulting these rate sheets (shown in Figs. 2, 3 and 4 and the accompanying tables) the approximate rates for any prospective consumer can be found after first ascertaining the aggregate rated capacity of his plant and the load factor. However, in preparing a contract, the exact rates are to be determined by substitution in the formulas.

TABLE I.—EQUIVALENT FLAT RATES, BASED ON FORMULA "B" IN DOLLARS.

Horse-power.	Load Factor— 35	40	45	50	60	70	80	100
25	42.50	46.14	49.78	53.43	60.72	68.00	75.28	89.86
30	41.23	44.76	48.30	51.83	58.90	65.97	73.03	87.17
40	39.39	42.67	46.04	49.40	56.14	62.88	69.62	83.09
50	37.86	41.10	44.35	47.59	54.09	60.58	67.07	80.05
60	36.73	39.88	43.03	46.17	52.47	58.77	65.06	77.66
75	35.39	38.42	41.46	44.49	50.56	56.62	62.69	74.83
100	33.73	36.62	39.51	42.40	48.19	53.97	59.75	71.32
125	32.50	35.29	38.07	40.86	46.43	52.00	57.57	68.71
150	31.53	34.23	36.93	39.64	45.04	50.45	55.85	66.65
175	30.73	33.36	36.00	38.63	43.90	49.17	54.44	64.97
200	30.05	32.63	35.20	37.78	42.93	48.08	53.23	63.53
250	28.96	31.44	33.92	36.41	41.37	46.34	51.30	61.23
300	28.09	30.50	32.90	35.31	40.13	44.94	49.76	59.39
400	26.77	29.06	31.36	33.65	38.24	42.83	47.42	56.60
500	25.80	28.01	30.22	32.43	36.86	41.28	45.70	54.55
750	24.11	26.18	28.24	30.31	34.44	38.58	42.71	50.98
1,000	22.98	24.95	26.92	28.89	32.83	36.77	40.71	48.59
1,300	22.00	23.89	25.77	27.66	31.43	35.20	38.97	46.51

TABLE II.—INVESTMENT AND MAINTENANCE CHARGES BASED ON FORMULA "C" IN DOLLARS.

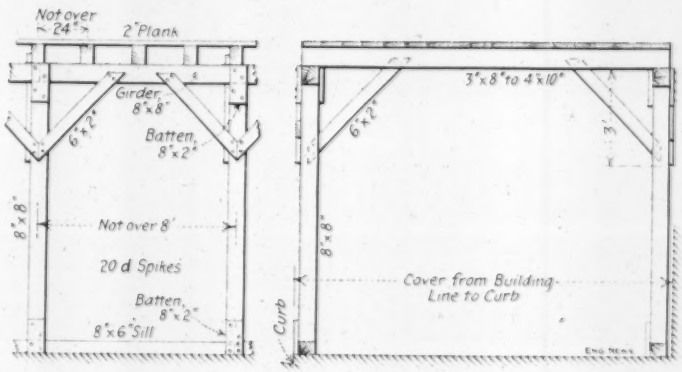
Horse-power.	Load Factor— 35	40	45	50	60	70	80	100
25	25.50	27.68	29.87	32.06	36.43	40.80	45.17	53.92
30	24.74	26.86	28.98	31.10	35.34	39.58	43.82	52.30
40	23.58	25.60	27.62	29.64	33.68	37.73	41.77	49.85
50	22.72	24.66	26.61	28.55	32.45	36.35	40.24	48.03
60	22.04	23.93	25.82	27.70	31.48	35.26	39.04	46.60
75	21.23	23.05	24.88	26.69	30.34	33.97	37.61	44.90
100	20.24	21.97	23.71	25.44	28.91	32.38	35.85	42.79
125	19.50	21.17	22.84	24.52	27.86	31.20	34.54	41.23
150	18.92	20.54	22.16	23.78	27.02	30.27	33.51	40.00
175	18.44	20.02	21.60	23.18	26.34	29.50	32.66	38.98
200	18.03	19.58	21.12	22.67	25.76	28.85	31.94	38.12
250	17.38	18.86	20.35	21.85	24.82	27.80	30.78	36.74
300	16.85	18.30	19.74	21.19	24.08	26.96	29.86	35.63
400	16.06	17.44	18.82	20.19	22.94	25.70	28.45	33.96
500	15.48	16.81	18.13	19.46	22.12	24.77	27.42	32.73
750	14.47	15.71	16.94	18.19	20.66	23.15	25.63	30.59
1,000	13.79	14.97	16.15	17.33	19.70	22.06	24.43	29.15
1,300	13.20	14.33	15.46	16.60	18.86	21.12	23.38	27.91

TABLE III.—METER RATES, BASED ON FORMULA "D" IN MILLS.

Horse-power.	Load Factor— 35	40	45	50	60	70	80	100
25	7.43	7.06	6.77	6.54	6.19	5.95	5.76	5.50
30	7.21	6.85	6.57	6.34	6.01	5.77	5.59	5.34
40	6.87	6.53	6.26	6.05	5.73	5.50	5.33	5.09
50	6.62	6.29	6.03	5.83	5.52	5.30	5.13	4.90
60	6.42	6.10	5.85	5.65	5.35	5.14	4.98	4.75
75	6.19	5.88	5.64	5.45	5.16	4.95	4.80	4.58
100	5.90	5.60	5.37	5.19	4.92	4.72	4.57	4.37
125	5.68	5.40	5.18	5.00	4.74	4.55	4.40	4.21
150	5.51	5.24	5.02	4.85	4.59	4.41	4.27	4.08
175	5.37	5.10	4.90	4.73	4.48	4.30	4.17	3.98
200	5.26	5.00	4.79	4.62	4.38	4.20	4.07	3.89
250	5.07	4.81	4.61	4.46	4.22	4.05	3.92	3.75
300	4.91	4.67	4.47	4.32	4.09	3.93	3.80	3.64
400	4.68	4.45	4.26	4.12	3.90	3.75	3.63	3.46
500	4.51	4.29	4.11	3.97	3.76	3.61	3.50	3.34
750	4.22	4.01	3.84	3.71	3.51	3.37	3.27	3.12
1,000	4.02	3.82	3.66	3.54	3.35	3.22	3.11	2.97
1,300	3.85	3.66	3.50	3.39	3.21	3.08	2.98	2.85

Protection Sheds Over Sidewalks.

In erecting city buildings it is usually necessary either to close off the sidewalk in front of the building or to cover the sidewalk with a protection shed for shielding passers-by from falling material. Such a shed to be effective must be substantially built, particularly where the structure being erected is high. It is the concern of the municipal building inspection authorities to see that the safety of the public is duly provided for in this respect, and in New York City therefore the Bureau of Buildings of Manhattan borough has recently drawn up a specification fixing minimum dimensions of parts for sidewalk sheds.



MINIMUM REQUIREMENTS FOR SIDEWALK SHEDS AS FIXED BY BUREAU OF BUILDINGS, MANHATTAN BOROUGH, NEW YORK CITY.

The drawing herewith gives the make-up and dimensions of the specified shed completely, except that the size of the cross-beams is made to vary with the sidewalk width and height of building as follows: For buildings over 65 ft.* but under 100 ft. in height, 3 × 8-in. beams when the sidewalk is not over 10-ft wide, and 3 × 10 when the sidewalk is over 10 ft. wide. For buildings over 100 ft. high, 3 × 10-in. beams when the sidewalk is not over 10 ft. wide, and 4 × 10, when the sidewalk is over 10 ft. wide. These are minimum sizes. When the shed roof is to be used for storage, or otherwise will have to carry heavy loads, the shed must be proportionately stronger; this condition is not at all rare.

The shed may differ in design from that shown, but must be of equal strength, and the plans must be approved by the Bureau.

The Factory Testing of Automobiles.†

The question, "How are automobiles tested by the builders," is one that mechanical engineers have often asked. The many engineering points involved in this design and operation of all sorts of motor cars makes the testing methods of more than passing interest. Therefore the methods employed at one factory (making the "Rambler" cars) seem worthy of notice. Because it is more difficult to detect flaws in a completed engine and a completed chassis than to discover weaknesses in separate parts, one entire building—covering floor space equivalent to one-half the floor area of the original factory—is devoted to motor and chassis testing alone. This is not all the testing done, of course, if one includes the rigid inspections of materials and workmanship which are necessary.

In the engine testing room 17 dynamos are arranged on concrete stands, each stand making a complete unit, with all necessary cooling apparatus and ducts to carry off exhaust gases. When a motor is completed in the engine assembly department, it is picked up on a small traveling crane, carried to the testing department and located on one of these testing stands, where it is first limbered up and then tested to establish its horse-power rating, and to detect any faults, before it is placed in the car.

During the limbering process the dynamo is run as a motor driving the engine. After this has continued a certain length of time, the engine is started and it drives the dynamo which then serves as load. The engine is run for several hours and, by ammeter and voltmeter readings, it can be easily determined whether or not the engine will develop its full rated capacity. During a loaded run, the engines are closely watched for noisy gears, valves or bearings. Before they leave these testing stands, they must be adjusted to run without knocking or undue vibration.

After an engine is assembled in a chassis, the partly completed car is returned to the testing department for a rear-wheel test. For this, the car is clamped with the rear wheels bearing on two large pulleys which just protrude through the floor. The shaft of these pulleys is driven by, or drives, a motor. The input into the motor, or the output from it while running as a generator, is controlled at the chassis by the tester and is indicated by instruments at his side. The control apparatus and instruments are mounted on conveniently located, permanent frames. This test insures the elimination of any error or poor adjustment in the transmission gear.

*The present building law requires sidewalk sheds only for buildings over 65 ft. high.

†From information furnished by Thomas B. Jeffery & Co., Kenosha, Wis.

The Groined Arch in Filter and Covered Reservoir Construction.*

By THOMAS H. WIGGIN,† Assoc. M. Am. Soc. C. E.

The groined arch for reservoir and filter roofs has been frequently described in good articles (as for example that by Mr. Leonard Metcalf in the Transactions of the American Society of Civil Engineers for 1900, with valuable discussions by W. B. Fuller and others), and the writer's only excuse for this paper is his participation or acquaintance with the design of a number of recent important plants in connection with which fresh consideration has been given to old problems. Incidentally some useful data, mainly drawn from other papers, but extended where convenient, are collected in the paper.

The simple beam, the continuous girder, the simple arch with parallel ends, the retaining wall and the suspension bridge, all handed down to us through generations of empirical engineering and architecture, succumbed a comparatively long time ago to the analysis of the theorist—at least so far as to yield passable hypotheses on which to tabulate the results of experience, thus permitting confident interpolation or extrapolation for new problems. The groined arch, although used centuries ago, remains a purely empirical type of structure, designed, or speaking more accurately, drawn without satisfactory mathematical analysis and used without much knowledge of the factor of safety.

The groined arch has long been used in the architecture of public buildings, probably at least since the second century of the Christian era, but its use in engineering structures has been greatly augmented in the last fifteen years by the advent of the covered slow sand filter plant and its attendant filtered-water reservoir, where the purified water is kept free from the algae-stimulating sun. To Mr. William Wheeler, of Boston, is due the credit for first using the groined arch in this country in waterworks construction, the initial plants embodying it being the filters at Somersworth, N. H., and Ashland, Wis.

The reasons other than precedent for this use of the groined arch are in part as follows:

(a) Absence of air pockets, thus permitting perfect ventilation.
(b) Economy and simplicity—no reinforcing being used for ordinary conditions.
(c) Uniform head room both longitudinally and transversely, thus permitting wires for lighting, and pipes or other devices for handling sand to be run level in either direction without waste of head room; also permitting economical lighting.

In 1903 Mr. Leonard Metcalf listed the groined arches in this country with their dimensions and other data, and that list was published in Engineering News, December 24, 1903. The accompanying Table I. is extended from that of Mr. Metcalf's, the additions including, however, only certain of the larger recent works.

The commonest form of groined arch roof is one in which the intrados is composed of intersecting semi-elliptical cylindrical surfaces and the extrados of intersecting parabolic cylindrical surfaces. The ellipses and parabolas are generally the same in both longitudinal and transverse arches, but are sometimes different when some special dimension is to be matched in each direction. The floors under piers are generally parabolic groins. The following formulas for volume of such groins were obtained from Mr. John H. Gregory, who worked them out with Mr. W. B. Fuller while they both were resident at the Albany filter plant for Mr. Allen Hazen.

Volumes of Elliptical Groined Arch Units

(See Fig. 1.)

m = vol. included between plane through top of pier and plane tangent to intrados at crown.
n = vol. of parallelopiped included between planes tangent respectively to intrados and extrados at crown, i. e., t apart vertically.
p = vol. included between extrados of arch and plane tangent to extrados at crown.

$$v = m + n - p.$$

$$m = 4b\left[cf + \frac{2ad}{3} - \frac{\pi}{4}(cd + af)\right]$$

$$n = 4\ fct.$$

$$p = \tfrac{2}{3}\ fch.$$

If the groin is square as is usual, $a = d$ and $c = f$ and the formulas reduce to the following:

$$m = 4b\left[c^2 + \frac{2a^2}{3} - \frac{\pi}{2}ac\right];$$

$$n = 4c^2t;$$

$$p = \tfrac{2}{3}\ c^2h.$$

The floor of filters and covered reservoirs has usually been made of groin units similar to those used in the roof but inverted, having parabolic instead of elliptical intrados, and having a flat instead of parabolic extrados because of resting on the ground. Mr. Gregory's formulas for volume of such units is as follows: (See Fig. 2.)

$$V = V_1 + V_2.$$

$$V_1 = 4b\left[cf - \tfrac{2}{3}(af + cd) + \frac{ad}{2}\right]$$

$$V_2 = 4cft.$$

If groin unit is square, that is, if $d = a$ and $f = c$

$$V_1 = 4b\left[c^2 - \tfrac{4}{3}ac + \frac{a^2}{2}\right]$$

$$V_2 = 4c^2t.$$

Diagrams by Mr. Gregory for quickly obtaining with good accuracy the volumes of any arches of given dimensions were published in Engineering News, Aug. 23, 1900, p. 131.

Mr. Gregory has also developed formulas and diagrams for the most economic shape of filter bed for use in subdividing a given area of filter surface into units of a given area.* The economy is largely a matter of diminishing length of dividing and marginal walls and is not directly concerned with the roof, which is fixed, except at sidewalls, by total area. There are generally conditions of local topography that fix the layout within rather close limits. Nevertheless it is useful to remember that shape of beds affects economy. In using Mr. Gregory's diagrams it is necessary to obtain the ratio between cost of dividing walls and cost of outside walls per foot. In the latter cost must be included extra filling both for refill and embankment, also extra excavation to permit placing and to provide for the extra embankment. With side walls of aqueduct section in ordinary cases this will result in a ratio of about 1 and the economical shape will have widths 6/10 to 8/10 of lengths if there are six or more beds.

(b) Piers have generally been square but a tendency is noticeable to make them circular. The circular pier has the advantage in construction that a thin sheet-iron form can be used. It also gives a more effective distribution of material in the column. It causes a little complication at its junction with the square base of the roof groin unit but this is not serious as the corners of the roof groin base can be easily chamfered just above the pier by the use of stucco blocks so as to fit the circular top of the pier. Piers of the earlier filters were enlarged by steps, in the filter sand, to prevent direct passage of the water without proper filtering. The more recent filters have flaring bases but not steps. Walls of filters have gone through a similar development, batters being used where there were formerly steps.

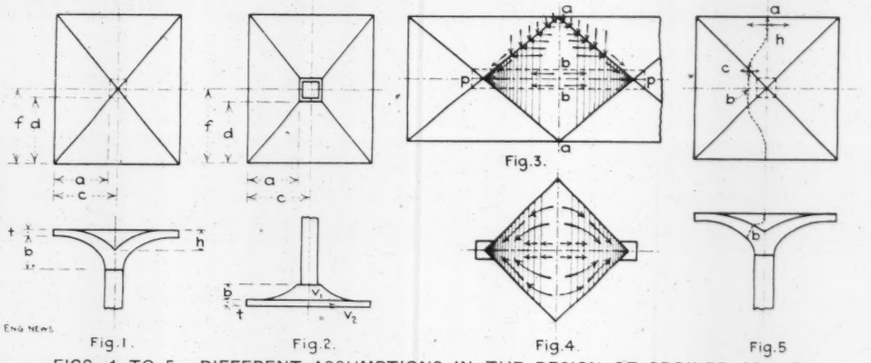


Fig.1. Fig.2. Fig.3. Fig.4 Fig.5

FIGS. 1 TO 5. DIFFERENT ASSUMPTIONS IN THE DESIGN OF GROINED ARCHES.
(In Fig. 5 the arrows at C should all be of same length. See text.)

Methods of Design

Returning now to the matter of design, there are several methods of considering the groined arch roof as a support for vertical loads:

A. The groined arch is often considered as a series of cylindrical arches (Fig. 3) cut off diamond-shape by the groin lines so that only those parts (*bb*) opposite the piers are full-centered, the balance being segmental of length constantly less as the intersections (*a*) of groin lines are approached, this length being zero at these intersections. The arch forces traveling down the segmental arches meet corresponding forces from adjacent arches at the groin lines at an angle of 90° and are supposed to have a resultant acting down the groin line into the pier. This method carries with it the thought that the arch should be thickened along the groin lines (*a p*) since otherwise there would be no arch to carry the diagonal resultants except what was filched from the adjacent segmental arches. Architecture has many examples of such thickening along the groin. Many of the engineers who design groined roofs, draw lines of resistance by the methods of graphical statics for the full-centered portion of the arch and assume that the segmental portions take care of themselves somehow down the groin lines without strengthening under the groin. Without an elastic theory for the groined arch it is impossible to compute with any mental satisfaction where the true line of resistance goes, even assuming the rigidity of abutments. Lines of resistance are generally drawn so as to start at the crown at a point distant from the center of thickness by 1/6 that thickness. This results in a maximum compressive stress twice as great as the average, in accordance with the well-known middle-third theory. Fig. 6 shows the result of a line of resistance drawn for the full-centered arch of the filtered water reservoir, Pittsburg, Pa.

B. The next method is best conceived of if one imagines the arch cut vertically along the groin lines (Fig. 4). The stresses in the segmental portion of the arch are conceived as converging gradually as the pier is approached. Or, in other words, the thrust from the segmental arches is conveyed over gradually to the full-centered part opposite the pier. Fig. 7 shows result by this method applied to the roof of the filtered water reservoir at Pittsburg, this being comparative with the result of Fig. 6 mentioned above. The crown thrust is about ¾ as great as in the barrel arch method.

C. The next method consists in considering each square groin unit as a cantilever or inverted dome just as if no bearing existed with the adjacent groin units (Fig. 5). These are really two methods as the dome idea places the groin unit entirely in tension whereas the cantilever has tension and compression on any vertical or approximately vertical section just as in a beam. The concrete in the top of the arch at a point such as *h* would seem to have tension in two principal directions at right angles to each other as shown by the arrows and of intensities having something like the ratio indicated by length of arrows. By the "theory of elasticity" these tensions at right angles would each reduce the effect of the other in rupturing the concrete. Similarly on the under side, the compression would aid each other. The magnitude of the effect due to similar stresses at right angles is considered as ⅓ of ¼; for example, a tensile strain in one direction will cause a compressive strain of ⅓ or ¼ its tensity, in a direction at right angles, and by that amount will diminish any tensile strain existing or tending to exist in this plane at right angles. The truth of this theory may be seen qualitatively by pulling in two directions on a wide rubber band. At point *a* stress apparently exists only in one direction and at a point like *c* on the groin line equal stresses in each direction would exist. It is hard to see how the reduction in maximum strain could be over 1/6 to ⅛.

As for the dome conception, the groin unit is not well adapted for such action, being square instead of round and requiring sharp changes of direction in ring stresses at the groin lines. Tests, to destruction, made under the writer's direction, of several groin units 15 ft. square and standing unsupported on all sides, showed that the initial crack always came at the middle of one of the sides. The weakest section, while not exactly determinable, is probably about as shown by line *a c b a*, Fig. 5. Fig. 8 shows a computation on this cantilever basis of the strength of the groin units of the filtered water reservoir, Pittsburg. The exact determination of all loads acting on the cantilever and their centers of gravity is too burdensome to be justified in so crude a method. Fairly close approximation can be estimated, however, as will be evident from the following explanations.

Computation of Groin Unit as Cantilever

The Pittsburg filtered water reservoir was planned for a filling varying from 4¼ ft. at the road in center down to about 3 ft. at the edges. It has actually only 3 ft. of filling. In table I. the loading is taken as 3 ft. at 110 lbs. + 15 lbs. of snow, following out Mr. Metcalf's methods; total load, 345 lbs. per sq. ft. On Fig. 8 and below, the results are carried out both for the actual loading on the above assumptions and for 4 ft. of filling at 100 lbs. plus 100 lbs. live load, the latter agreeing more nearly with assumptions used in design and also making the figures comparative with those of Mr. Casler's arch computations given on Fig. 7. The loads and lever arms referred to the assumed surface of fracture are as follows:

(See Table I. for General Dimensions.)

Weight of Concrete: ½ the groin unit less that part at pier within line of fracture. Average thickness, 8.2 ins. Wt. of ½ groin $\frac{8.2}{12} \times 18 \times 9 \times 150$ lbs. = 16,600

Wt. at pier to be deducted (dimensions scaled from Fig. 8; irregular figure assumed to be a trapezoid):

Upper base 1.7 × 6	= 10.2
4 × midsection = 4 × 1.6 × 3.2	= 20.5
Botton midsection = 1.125 × 2.25	= 2.5
$\frac{33.2 \times 2.8}{6} \times 150$	= 2,300
Net dead weight outside of surface of fracture	14,300

Lever arm (eye estimation), 3.75 ft. = 45 ins.

*Paper presented at the Annual Meeting of the National Association of Cement Users, Chicago, Ill., Feb. 21-26, 1910.

†Senior Designing Engineer, New York Board of Water Supply, 299 Broadway, New York City.

*See Eng. News, Oct. 11, 1900, p. 252.

TABLE I.—DATA ON GROINED ARCH ROOFS FOR FILTERS AND RESERVOIRS

First 27 References Compiled by Leonard Metcalf (see Journal of New England Water Works Association); Remainder Compiled by Morris Knowles and T. H. Wiggin.

Reference No.	Date	Location	Engineer	Description	Depth of Earth Cover, ft.	Depth of Earth Cover, ins.	Concrete per cu. yd.	Arch Span, ft.	Arch Span, in.	Arch Rise, ft.	Arch Rise, in.	Thickness Crown	Thickness Haunches	Pier Section at Springing Line. Inches = square feet	Vol. Saved by Depress. in cu. yds. and Eq. Dph. in cu. yds. ins.	Load per sq. ft. Mean	Load per sq. ft. Crown	Pier Load, tons per sq. ft. above Springing Line Concrete at 150 lbs. Brick 130. Earth 110 and Snow 15	Intrados Rise (Span)	Extrados Rise (Span) + (Rise) (Thickness)
1	1895	Ashland, Wis.	Wm. Wheeler.	Filters, net area ½-acre	2	0	1:2½:5	15	9	3	6	6	None	{28 × 23 = 4.42 {28½ × 24 = 4.73 brick	None	11.7	....	12.5	8.9	10.6[1]
2	1896	Somersworth N. H.	Wm. Wheeler.	Two filters, total net area ½-acre	2	6	1:2½:5	16	0	4	0	6	None	34½″×34″ = 8.15 gran.	None	13.0	0.47	9.7	8.0	10.0[2]
3	1897	Wellesley, Mass.	Freeman C. Coffin.	Reservoir, 80 feet int. diameter	2	0	1:2½:5	12	0	2	6	6	None	24″ × 24″ = 4.0 brick.	None	10.3	0.225	8.9	7.2	8.5
4	1898	Louisville, Ky.	Chas. Hermany.	Res. 154,739 sq. ft. net Ar., steel rein.-con.	6		1:2:4	19	0	3	9.6	6	27.6	Dia. 3.4′ = 9.0	5.45 = 3.7	11.7	0.61	5.8	(11.9)	(13.8)[3]
5	1899	Concord, Mass.	Leonard Metcalf.	Sewage reservoirs, 57 ft. int. diameter	2	6	1:2:5	12	9	3	0	6	None	24 × 24 = 4.0 brick.	None	11.0	0.38	11.7	6.8	8.3
6	1899	Albany, N. Y.	Allen Hazen.	Filters, 8 beds, 0.7-acre each	2	0	1:3:5	11	11	3	6	6	6	21 × 21 = 3.06 brick.	0.93 = 1.63	8.5	{0.182 {0.277	11.0	7.1	8.4[4]
7	1899	Clinton, Mass.	F. P. Stearns.	Sewage reservoir, 100 ft. int. diameter	4	6	1:2½:4	12	0⅞	2	6	12	None	{30 × 30 = 6.25 {25 × 25 = 4.33 brick	None	16.8	....	12.2 11.7	7.3	8.5
8	1900	Superior, Wis.	Allen Hazen.	Filters and reservoir	2	0	1:3:5	12	0	2	6	6	6	20 × 20 = 2.77 brick.	0.93 = 1.63	8.4	0.29	12.0	7.2	8.5
9	1901	Phila. Pa., L. Roxbor'gh	John W. Hill.	Filters and reservoir	2	0	1:3:5	14	0	3	0	6	21	22 × 22 = 3.36	2.70 = 3.5	7.3	....	13.3	8.2	9.7
10	1902	Milford, Mass.	Leonard Metcalf.	Filters, two of ⅛-acre each	1	6	{1:2½:5 {1:3:5	14	0	3	0	6	18	24 × 24 = 4.0	2.37 = 3.0	7.9	0.36	9.8 16.8	8.2	9.7[5]
11	1902	Phila. Pa., U. Roxbor'gh	John W. Hill.	Filters and reservoir	2	0	1:3:5	14	0	3	0	6	21	22 × 22 = 3.36	2.70 = 3.5	7.3	....	13.3	8.2	9.7[6]
12	1902	Natick, Mass.	Frank L. Fuller.	Reservoir	2	0	1:2:4½	13	6	2	9	6	None	20 × 20 = 2.77	None	10.4	0.40	15.1	8.3	9.7
13	1903	Ithaca, N. Y.	Allen Hazen.	Coagulant basin and reservoir	2	0	{1:2½:4½ {1b.96f.11c.	10	6	1	6	6	10¼	{18 × 18 = 2.24 {17 × 17 = 2.01	0.75 = 1.70	6.78	....	10.9 & 12.2	9.2	9.9
14	1903	Proposed for Lawrence, Mass.	Morris Knowles.	Filter	3	0	.....	13	2	2	9	6	18	22 × 22 = 3.36	2.08 = 3.0	7.5	....	15.6	7.9	9.2
15	1903	Yonkers, N. Y.	Allen Hazen.	Reservoir	4	0	1:2.9:5	10	0	1	6	6	9¾	16¾ × 16¾ = 1.96	0.65 = 1.6	7.0	....	18.5	8.3	9.1
16	1903	Watertown, N. Y.	Allen Hazen.	Coagulant basin and reservoir	2	0	1:2.9:5	10	0	1	6	6	10	18 × 18 = 2.25	0.68 = 1.7	6.9	....	9.9	8.3	9.1
17	1903	Brookline, Mass.	F. F. Forbes.	Reservoir, all concrete	2	0	1:2:4	12	0	2	6	6	None	20 × 20 = 2.77	None	10.1	0.40±	12.2	7.2	8.4
18	1903	Phila., Pa., Belmont.	John W. Hill.	Filters, 18 of 0.74-acre each	2	0	1:3:5	13	5	3	0	6	21	22 × 22 = 3.36	2.51 = 3.5	7.3	....	12.4	7.2	9.0
19	1903	Phila., Pa., Belmont.	John W. Hill.	Filtered water basin, 382′ × 396′	2	0	1:3:5	14	0	3	0	6	21	22 × 22 = 3.36	2.70 = 3.5	7.2	....	13.3	8.2	9.7
20	1903	Phila., Pa., Torresdale.	John W. Hill.	Filters, 33 of 0.75-acre each	2	0	1:3:5	14	0	3	0	6	21	22 × 22 = 3.36	2.70 = 3.5	7.2	....	13.3	8.2	9.7
21	1903	Phila., Pa., Torresdale.	John W. Hill.	Filters, 22 of 0.75-acre each	2	0	1:3:5	13	2	3	0	6	21	22 × 22 = 3.36	2.48 = 3.5	7.4	....	12.0	7.2	8.7
22	1903	Phila., Pa., Torresdale.	John W. Hill.	Filtered water basin, 602′ × 762′	2	0	1:3:5	14	0	3	0	6	21	22 × 22 = 3.36	2.70 = 3.5	7.2	....	13.3	8.2	9.7
23	1903	Washington, D. C.	Col. A. M. Miller.	Filters Nos. 1 to 24	2	0	1.2.9:5	12	2	2	6	6	17	22 × 22 = 3.36	1.70 = 2.8	7.3	0.31+	9.7 + sand stored	7.4	8.6
24	1903	Washington, D. C.	Col. A. M. Miller.	Filters Nos. 25 to 29	2	0	1:2.9:5	11	10	2	6	6	17	22 × 22 = 3.36	1.63 = 2.8	7.4	0.31+	9.1 + sand stored	7.0	8.2[7]
25	1903	New Milford, N. J.	Hering & Fuller.	Clear water well under mech. filters	None		.....	9 11	8} 8}	2	2	6	8	None	24 × 24 = 4.0	None	11.7	....	30±	{4.7 {6.8	5.5[8] 8.1
26	1903	New Milford, N. J.	Hering & Fuller.	Clear water well under mech. filters	None		.....	11 12	8} 0}	2	6	8	None	24 × 24 = 4.0	None	12.3	....	30±	{6.8 {7.2	8.1[8] 8.4
27	1903	Washington, D. C.	Col. A. M. Miller.	Pure water reservoir	2	0	1:2.9:5	15	6	3	6	6	24¾	30 × 30 = 6.25	4.0 = 1.5	7.9	0.31+	8.7 + storage	8.6	10.3
28	1907	Wilmington, Del.	Theo. A. Leison.	Filters, 6 of 0.6-acre each	2	0	.....	13	6	3	0	6	21	Dia. 24 = 3.14	2.59 = 3.5	7.4	....	13.78	7.6	9.1
29	1907	Wilmington, Del.	Theo. A. Leison.	Filtered water reservoir	2	0	.....	13	6	3	0	6	21	Dia. 24 = 3.14	2.59 = 3.5	7.4	....	13.78	7.6	9.1
30	1903	N.Y.C., Stormville (Prop)	Wm. B. Fuller.	Filters, 200 of 0.97-acre each	†2	0	1:3:5	15 15	0} 10}	3	4	6	22	20 × 20 = 2.77	3.3 = 3.7	7.3	....	18.7	{8.4 {7.5	12.5 10.4
31	1903	N. Y. City, Hillview "	Wm. B. Fuller.	Filtered water reservoir	†2	4	1:3:5	20 12*	0 10}	4 *2	0 3	6 *8	22 None	Dia. 28 = 4.27 *20 × 20 = 2.77	5.64 = 3.6 None	8.5 ...	....	24.04	10.1 {9.1 {5.5	14.5 10.2 6.6
32	1906	Columbus, Ohio.	Julian Griggs.	10 filters and reservoirs	None		1:2½:5½	10 9	0} 5⅔}	2	6	8	None	21 × 21 = 3.06	None	12	....	About 29	{4.5 {6.0	5.75 7.35
33	1905	Cincinnati, Ohio.	G. H. Benzenberg.	Supports for 28 filters of .04-acre each.			.....	11	0											
34	1907	Jerome Park, N. Y.	I. M. De Varona.	Filters, 104 of 0.4-acre each	2	0	.....	13	9	2	0	6	*18	18 × 18 = 3.25	2.15 = 2.7	6+	....	18.91	12.6	9.0
35	1907	Jerome Park, N. Y.	I. M. De Varona.	Filtered water reservoir	2	0	.....	18	0	4	6	6	*21	Dia. 24 = 3.14	4.28 = 5.0	9.2	....	22.40	12.5	11.2
36	1906	Providence, R. I.	Ottis F. Clapp.	Filters	2	0	.....	10	4	2	6	6	*15	20 × 20 = 2.77	1.02 = 2.3	7.7	....	8.72	7.4	13.3
37	1907	New Orleans, La.	Geo. G. Earl.	Two filtered water reservoirs	1	0	.....	10*	9	2	6	6	*15	15 × 15 = 1.56	1.11 = 2.5	6.2	....	9.83	5.7	7.0
38	1901	Pittsburg, Pa.	Morris Knowles.	Filtered water reservoir	2	0	1:3:5	13	9	3	0	6	12	Dia. 27 = 3.98	1.58 = 2.0	9.0	....	11.8	7.8	9.4[9]
39	1904	Pittsburg, Pa.	Morris Knowles.	Filters, 46 of 1 acre each	2	0	1:3:5	13	2	2	9	6	21	22 × 22 = 3.36	2.43 = 3.4	7.3	.189	15.7	7.2	8.7[9]
40	1904	Pittsburg, Pa.	Morris Knowles.	Filtered water reservoir	2	0	1:3:5	15	2	4	3	6	26	Dia. 27 = 3.98	4.34 = 4.35	8.2	.204	17.6	7.7	9.7[9]
41	1908	Pittsburg, Pa.	Morris Knowles.	Filters, 10 of 1 acre each	2	3	1:3:5	13	2	2	9	6	21	Dia. 20 = 2.18	2.28 = 3.25	7.3	.276	19.4	7.2	8.7[9]
42	1908	Springfield, Mass.	Elbert E. Lochridge	Filters, 6 of 0.52-acre each	2	0	.....	11	4	2	6	6	15	20 × 20 = 2.77	1.30 = 2.1	6.6	....	9.65	8.0	8.9
43	1908	Springfield, Mass.	Elbert E. Lochridge	Filtered water reservoir	2	0	.....	14	0	2	9	6	18	24 × 24 = 4.0	2.36 = 3.0	8.5	....	10.52	8.5	10.3
44	1909	N. Y. City, Hillview.	J. Waldo Smith.	Filtered water reservoir (proposed)	†18″ to 3′		1:3:5	19	2	2	5	6	31	Dia. 34 = 6.3	9.25 = 6.2	8.22	....	13.1 to 19.4	9.2	11.7

*Note: These dimensions were scaled. †Will not be built.
[1]Cost column covers cost masonry only from springing line of roof.
[2]Earth cover and engineering not included unless so stated.
[3]Earth cover, etc., included total exclusive engineering only (10%±). See text. Steel Reinforcement.
[4]Including filling, manholes, etc., but excluding engineering.
[5]Cost per square foot is estimated from force accounts. Contractor's price was less. (Poor management.)
[6]Including sand stored upon roof.
[7]Four-fifth of clear well with these arches.
[8]One-fifth of clear well with these arches.
[9]Carries filters. About 1,100 lbs. per sq. ft. Reinforced with ⅝-in. square twisted bars. Spaced 12 ins. in both directions.
[10]Cost includes concrete and centering for bay without ventilator.
[11]Cost includes concrete and centering for bay without ventilator.
[12]Cost includes concrete and centering for bay without ventilator.

Earth Filling in Hollow Over Pier:
Total quantity = 4.34 cu. yds. (see Table I.).
From ½ of 4.34 yds. subtract part within line of fracture. (Dimensions scaled from Fig. 8; irregular figure assumed to be a trapezoid).

Upper base	½ × 4.34 × 27 at 100 lbs.	=	5,859
4 × mid section	6 × 1.5 =	=	9.0
Lower base	4 × 5.6 × 1.4 =	=	31.4
	40.4 / 6 × 2.17	=	14.6
Weight of earth filling in hollow and outside of surface of fracture.	at 100	=	1,460
(2d assumption) Weight of same if assumed 110 lbs. per cu. ft.			4,400

Lever arm (eye estimation) 2¼ ft = 27 ins.

Four feet of earth filling at 100 lbs. above plane through extrados of crown plus 100 lbs. liver load.
Part outside of surface of fracture = (18 × 9.5 × 1.5) at 500 lbs. = 78,500
(2d assumption) 3 ft. of earth filling at 110 lbs. × × 1.5) at
plus 15 lbs. snow load = (18 × 9.5 × 1.5) at = 52,800
345 lbs.

Lever arm estimated at 4 ft. = 48 ins.

The tensile strength at the critical point at the top of the groin may be obtained by the usual formula for beams, $f = \frac{My}{I}$,

where f = the stress desired.
M = the moment, in in.-lbs., of the loads.
y = the vertical distance from the center of gravity of the surface of fracture to the point where the stress is desired.
I = the moment of inertia of the surface of fracture. In the computations the projection of this surface is used and lever arms are measured from a point about 1 ft. out from the center of pier. Approximations are unavoidable.

The computations and results are shown in tabular form on Fig. 8. It will there be seen that the dead load of the concrete alone results in a stress of about 65 lbs. per sq. in., while the 500 lbs. per sq. ft. and 345 lbs. per sq. ft. loadings above the extrados result in 490 and 340 lbs. tension, respectively. It is therefore not surprising that the test groin units, standing alone, break of their own weight or with at most with a load of 200 lbs. per sq. ft.

Effect of Steel in Groin Unit.

A natural suggestion for reinforcing a groin unit is to place a ring of steel around the unit in the manner indicated in Fig. 8. Such a device was used at Louisville (line 4, Table I.). There are several ways of estimating quantitatively the effect of such steel.

(a) Consider that it assists the concrete in the upper part of the groin unit in carrying tension. It would then be necessary to add enough steel to bring the tension in the concrete down to a conservative unit, say 50 lbs. per sq. in. The steel could not be at the very outside fiber ($y = 1.25$), but might be, say, 1 ft. from the center of gravity of the concrete section. The neutral axis would be raised very much if enough steel should be added to the tension side so that the full dead and live loads might be carried with only 50 lbs. tension in the concrete. This would diminish the area of tension concrete and increase the steel needed. But assume, even, that the neutral axis does not rise; the steel would have

$$\frac{Es}{Ec} \times \frac{1.25}{1} \times 50 = \text{say } 500$$

lbs. per sq. in. tension when the concrete had 50 lbs. There is approximately 3.1 sq. ft. of concrete surface above the neutral axis in the half-section of fracture and under the 500 lbs. per sq. ft. loading tabulated on Fig. 8, the tensile stress on it varies from 0 to 460 lbs. per sq. in. or, roughly, 230 lbs. average. To reduce this stress so as to make it vary from 0 to 50 as above assumed, or 25 lbs. average, would require, by this rough method of computing, the quantity of steel, x sq. ft., as shown by the following:

$$144\,[x \times 500 + (3.1 - x)\,251 = [31.1 \times 250]\,144;\ x = 1.3.$$

That is, 1.3 sq. ft. of concrete on the tension side of

each half section would have to be replaced by steel to bring the tension in the concrete down to 50 lbs. per sq. in. If proper account were taken of the rising of the neutral axis, the result would be considerably in excess of 1.3 sq. ft., but this result is sufficient to show the absurdity of the conception, which is treated at this length only because it is so often assumed that *somehow* by adding a little steel joint action will result between the steel and concrete in tension, and at nominal expense make a structure which will stand any abuse safely.

(b) Assume that the concrete takes no tension and that the compression on it may be as high as 600 lbs. per sq. in—i. e., the usual reinforced-concrete beam theory. All warranted accuracy in computing the steel by these assumptions will result from estimating the "centroid" of compressive stresses and using the lever arm between this centroid and the steel. For the full groin loading the neutral axis will drop and the centroid likewise from their positions without steel. The centroid without steel is about 1 ft. below the neutral axis and the outer fiber on the compressive side 1.4 ft. Assume 1.15 ft. as the distance of the new centroid; 1.05 ft. is the greatest practicable distance for the steel above the neutral axis. The steel hence has a lever arm of 1.15 + 1.05 = 2.2 ft. For a moment of 4,440,000 in.-lbs. the steel would have to carry $\frac{4,400,000}{2 \times 2.2 \times 12} = 83,000$ lbs. and of mild steel 5 to 7 sq. ins. would be needed. Tests have shown that lapping such steel is not successful on account apparently of the thin section of concrete splitting. Such reinforcement is hard to place satisfactorily around the corners as the extrados is depressed along the groin line so that the steel has to be kept out near the corner and hence turned sharply (see Fig. 8). This brings a heavy compressive stress on the concrete at the inside of the curve in the steel. The steel will apparently have about ⅔ as much tension at the groin lines as at the middle of the sides, or, say, 12,000 lbs. per sq. in. Assume a radius of 18 ins. for a bar 1½ ins. square as in the contract plans of the Pittsburg filtered water reservoir. Further, assume the stress is carried around the corner by uniform radical pressure as in cylinder under hydraulic pressure.

$$1.5^2 \times 12,000 = 27,000 \text{ lbs. in the bar.}$$

The bearing pressure of the steel against the concrete would then be $\frac{27,000}{1.5 \times 18} = 1,000$ lbs. per sq. in.

Enough has been written, the writer thinks, to eliminate cantilever or inverted dome action as the final reliance in carrying load. Arch action is adequate without aid from cantilever action.

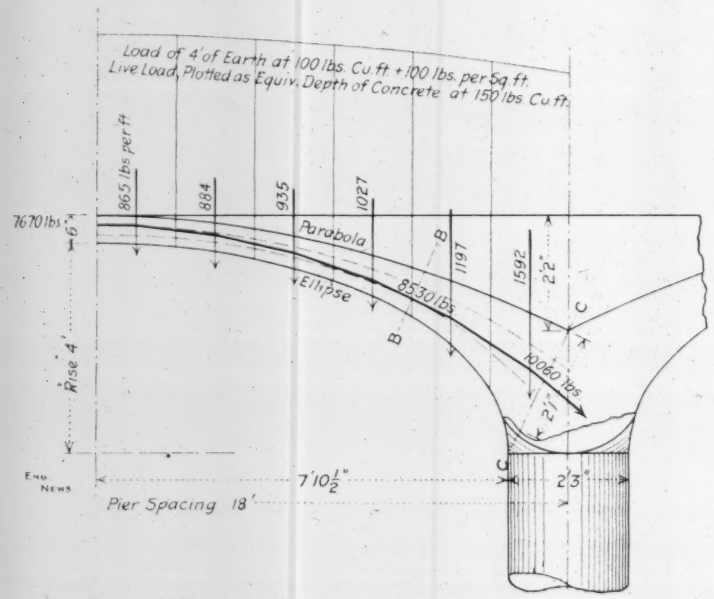


FIG. 6. METHOD OF COMPUTATION ASSUMING STRESSES SAME AS IN BARREL ARCHES.

(As applied to Filtered Water Reservoir, Pittsburg).

TABLE OF STRESSES IN ARCH.

Location of section.	Thrust per lin. ft. of arch.	Area in sq. in.	Av. stress in lbs. per sq. in.	Max. stress in lbs. per sq in.
Crown	7,670*	6 × 12	106.5	213
B B	8,530	10 × 12	71.	142
C C	10,060	25 × 12	33.5	...

*Crown thrust for whole groin unit = 7,670 × 18 = 138,000 lbs.

Shrinkage and Temperature Changes.

But the problem is not so simple as it seems from the foregoing academic discussion of the arch theories. The disturbing elements are two:

(a) Shrinkage of concrete in setting.

(b) Expansion and contraction of concrete with change of temperature.

Groined arch roofs are usually laid in strips of random length and as they set it is frequently possible to see upward between the strips. Less shrinkage takes place if sprinkling is faithfully done and the earth covering is put on early.

As for temperature, the concrete will usually be subjected, before the filters or reservoirs are put in service, to extremes from 90° to say 15° F. or lower. This means a change in length of ⅝-in. in every 100 ft., or approximately ⅛-in. for every groin unit. To keep the arch action in those groins in which the concrete cannot endure a high tension in cantilever action (see Fig. 8 for magnitude of tension), it is apparently necessary for cracking to occur sufficient to move each side of the groin outward from the pier a distance of about 1/16-in., *less* the slight movement resulting from the elasticity of the concrete when acting as a cantilever. If this cracking took place along the line of fracture assumed on Fig. 8 it would be visible at the crown on the inside but probably not elsewhere from the inside without minute examination. Such cracks are occasionally seen but no one has ever excavated on the outside so far as the writer knows to trace them for their full length. They would exist in their maximum size only in the coldest weather before the reservoir or filter was closed for service. Examinations are not usually made at such times in any unnecessary cases. After the works are in service the temperature would rarely be below 40° F. and lesser cracks only would exist.*

Aid Given by Earth Covering.

In all these considerations, one factor has been neglected, viz.: the value of the earth cover. Ordinary earth filling does not crack with reduced temperature but more often expands due to freezing of water in the interstices. Mr. F. H. Robbins has pointed out that a very few pounds per square foot in the earth cover would transmit the crown thrusts without help from the concrete. If this action occurred the concrete would be held generally along its top surface by friction and cracks might be distributed. The earth filling the depression would in any case tend to hold the sides of the groin units away from the pier. No proved conclusion on the temperature matter can be drawn except that every groined roof that has not been admittedly maltreated in construction, by allowing the concrete to freeze or pulling the centers prematurely, has stood successfully not only its uniform load from fill, but sundry extra loads, such as piles and heavy construction material.

Construction Stresses.

During the construction of groined arch roofs it is occasionally necessary to leave a strip of roof uncompleted so that a space intervenes between the completed roof and the side or end wall of the filter or reservoir. This leaves the groined arches without abutment and at once raises the question as to whether the forms may be removed and earth filling placed also. Construction foremen will tell tales of long edges exposed with the fill completely placed, but the writer has always been unable to substantiate these statements from the engineers in charge. At the Pittsburg filters long strips of roof had to be omitted to form lanes through which the cableways could operate, but where filling was to be placed on the roof diagonal bracing was required strong enough to transmit the thrust to the floor: that this bracing was a wise precaution is attested by the fact that in spite of it some of the piers were lifted 1/16-in. or so from their bases on the side away from the braces and another opening took place at the top of the pier. Pictures are shown of teams or moderately heavy rollers going along close to the edge of a void the size of two or three groin units. When it is considered that such loads cover an area of perhaps 50 sq. ft. and are scarcely equivalent in weight to the earth which would be on that small area; further, that the groin is usually entirely free from its earth load in such cases; further, that friction between groins would carry the stress to the groin at the end of the void space, it would be indeed surprising if the arch did not easily endure the load.

*Since writing this the writer has found many evidences that cracking does occur in about the position shown on Fig. 8. These evidences were found in a very few minutes' examination of one filter and a short length of gallery. The cracks are generally straighter than the line of fracture shown on Fig. 8.

It is recognized in construction that earth filling may be safely placed within a reasonable distance, depending on the height of roof, of an incomplete edge of filter or reservoir groined roof and it is pertinent to inquire what the limits of safety are. The only resisting forces are the cantilever action in the groin units and the stability of the piers weighted with their superimposed groins. Two cases may be considered.

Case I. Earth loading on roof terminated abruptly.

Let n = no. of empty groins with piers that are necessary to resist the thrust from the loaded portion of roof.

Then for the filtered water reservoir at Pittsburg (Fig. 9)

Wt. of 1 pier $= 3.98' \times 21.5 \times 150 = 12,800$ lbs.

Lever arm of pier $= \frac{2.25'}{2} - .17 = .95$ ft.

Wt. of groin unit $= \frac{8.2}{12} \times 18^2 \times 150 = 33,200$ ft.

Lever arm of groin unit $= 2.25 - .33 = 1.92$ ft.

(This assumes that there is enough friction and continuity in the roof at the crown points to keep it in its original plane and not permit tipping. Otherwise the lever arm would be only half as great. Observations bear out the assumptions.)

(a) Assume thrust as in Fig. 7 $= 105,200$ lbs.

Lever arm $= 21.5$ ft.

Then n $(12,800 \times .95 + 33,200 \times 1.92) = 105,200 \times 21.5$

$75,900\ n = 2,260,000$

$n = 30$. That is, it would take 30 groin units and piers, or 540 ft. of unloaded reservoir roof, to resist the overturn due to a full load on the adjacent roof.

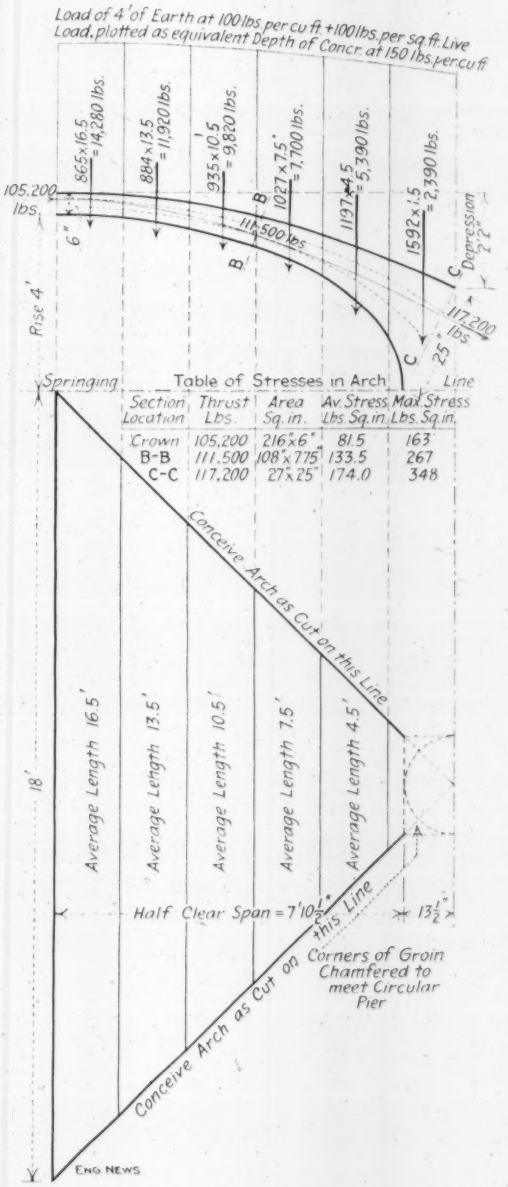

Load of 4' of Earth at 100 lbs per cu ft + 100 lbs per sq ft. Live Load, plotted as equivalent Depth of Concr. at 150 lbs per cu ft
865 x 16.5 = 14,280 lbs.
884 x 13.5 = 11,920 lbs.
935 x 10.5 = 9,820 lbs.
1027 x 7.5 = 7,700 lbs.
1197 x 4.5 = 5,390 lbs.
1592 x 1.5 = 2,390 lbs.
105,200 lbs.
111,500 lbs.
117,200 lbs.
6"
Rise 4'
2'2" Depression
2'5"
Springing Line
Table of Stresses in Arch
Section Location | Thrust Lbs. | Area Sq. in. | Av. Stress Lbs Sq in | Max Stress Lbs Sq in.
Crown | 105,200 | 216 x 6" | 81.5 | 163
B-B | 111,500 | 108 x 7.5" | 133.5 | 267
C-C | 117,200 | 27 x 25" | 174.0 | 348
Conceive Arch as Cut on this Line
18'
Average Length 16.5'
Average Length 13.5'
Average Length 10.5'
Average Length 7.5'
Average Length 4.5'
Half Clear Span = 7'10½"
13½"
Conceive Arch as Cut on this Line
Corners of Groin Chamfered to meet Circular Pier
ENG NEWS

Fig. 7. Method of Computation Assuming Arch Cut on Groin Lines.

(Filtered Water Reservoir, Pittsburg, Computation by M. D. Casler.)

Whatever strength the groin units have as cantilevers, including any effect of continuity of several units, would constitute the only margin of safety. If it were not desired to risk the groins supporting their own weight as cantilevers, it would be necessary to leave the forms under the roof for a sufficient distance from the edge so

that the stability of the supported groins would take the thrust from an unsupported unloaded groin. It is customary to thus leave forms in place. The number required to be supported by forms will appear from Case II. below. It should be noted that any part of the weight of roof that is carried on wooden posts supporting forms is not available to resist overturn. The weights of forms coming on piers may be assumed to offset this deduction.

Case II. Assume earth loading to be tapered off toward the edge (Fig. 10). This will permit full loading nearer the edge than in Case I., since the partial earth loading increases the stability. Even a reasonable estimate under the assumptions requires considerable detailed computation.

Data: (See Case I. for certain of results below listed.)

1. Weight of 1 pier = 12,800 lbs.
2. Assumed lever arm of pier = .95 ft.
3. Weight of 1 groin unit = 33,200 lbs.
4. Assumed lever arm of groined units (also of any loading on groin unit) = 1.92 ft.
5. Resistance to overturn of 1 pier and groin unit = 76,000 ft.-lbs.
6. Wt. of filling in hollow of groin 4.34 cu. yds., at 2,700 = 11,700 lbs.
7. Resistance to overturn afforded by filling of hollow in groin unit = 11,700 × 1.92 = 22,000 ft.-lbs.
8. Wt. of 1 ft. of filling at 100 lbs. or of 100 lbs. per sq. ft. live load 18 × 18 × 100 = 32,400 lbs.
9. Resistance to overturning afforded by 1 ft. filling = 32,400 × 1.92 = 62,000 ft.-lbs.
10. Load on one groin unit due to 4 ft. of filling and 100 lbs. per sq. ft. live load = 500 × 18 × 18 = 162,000 lbs. Thrust with 500 lbs. sq. ft. load (see Fig. 7) = 105,200 lbs.

Assumptions:

The thrust of 105,200 lbs. is due to the dead load, earth load and live load roughly in proportion to weight of each load, but is more closely proportioned on the assumption that it is due to

(a) *8/10 of dead weight of groin unit;

(b) ½ of filling in hollow of groin unit;

(c) The whole of filling and live load above plane through extrados at crown.

(The use of 8/10 and ½ the loads in (a) and (b) is due to the fact that a part of weight of groin unit and filling in hollow is concentrated near pier.)

(a_1) 8/10 × 33,200 = 26,600
(b_1) ½ × 11,700 = 6,000
(c_1) 1 × 162,000 = 162,000

194,600

causes thrust of 105,200 lbs.

(a_2) $\frac{26,600}{194,600} \times 105,200 =$ *14,400 lbs. thrust due to weight of given unit.

(b_2) $\frac{6,000}{194,600} \times 105,200 =$ 3,200 assumed to be due to filling in hollow.

(c_2) $\frac{32,400}{194,600} \times 105,200 =$ 17,500 assumed to be due to each foot of filling or 100 lbs. sq. ft. live load above crown.

(a_3) 14,400 × 21.5 = 310,000 ft.-lbs. Overturning moment assumed due to wt. of unit.

(b_3) 3,200 × 21.5 = 69,000 ft.-lbs. Overturning moment assumed due to filling in hollow.

(c_3) 17,500 × 21.5 = 375,000 ft.-lbs. Overturning moment assumed due to 100 lbs. sq. ft.

(d_3) 105,200 × 21.5 = 2,260,000 ft.-lbs. Overturning moment assumed due to total load.

The computation in the table results in permitting the assumed full loading over the 18th pier from the edge. The table shows also that a filling of 3 ft. such as was actually used could be permitted over the 15th or 16th pier. Such relatively elaborate computations and applications of them have not been made in practice so far as the writer knows. Reservoirs of such large area exist at only a few places and the *filter* roofs are more stable by reason of lower and broader-based piers. Dividing walls also introduce stability about every 100 or 200 ft.

There are two cases to be considered as before. The figure (Fig. 11) is drawn for Case II., but, taken in connection with Fig. 9, will be applicable to Case I. also.

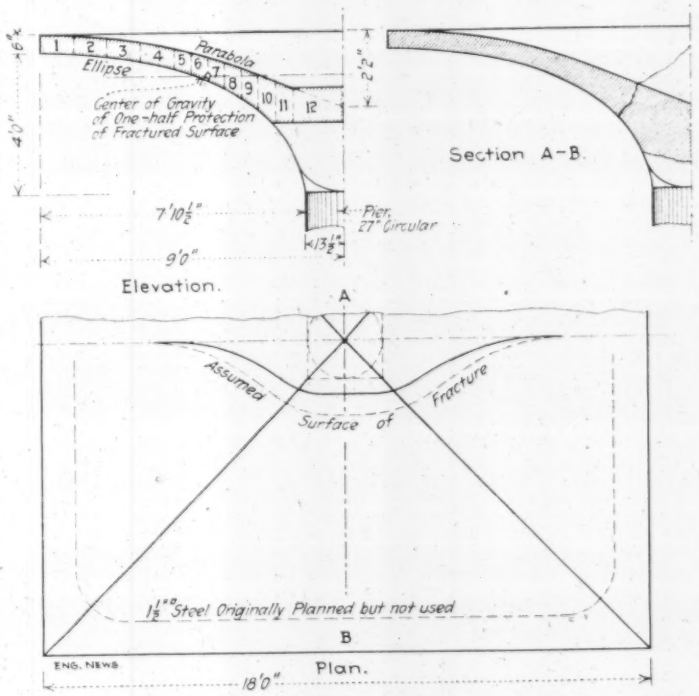


FIG. 8. METHOD OF COMPUTATION ASSUMING CANTILEVER ACTION. (Filtered Water Reservoir, Pittsburg.)

TABLE A.—COMPUTATION OF MOMENT OF INERTIA OF ASSUMED SURFACE OF FRACTURE.

No. of section.	b	h	I_o	a	g	ag^2
1	1	.52	.012	.52	1.0	.52
2	1	.53	.012	.53	.90	.43
3	1	.55	.014	.55	.78	.34
4	1	.57	.015	.57	.60	.21
5	.5	.61	.009	.30	.40	.05
6	.5	.65	.011	.33	.25	.02
7	.5	.70	.014	.35	.08	.0
8	.5	.80	.021	.40	.10	.0
9	.5	.90	.030	.45	.40	.07
10	.5	1.1	.055	.55	.63	.22
11	.5	1.05	.048	.53	.75	.30
12	1.5	1.02	.125	1.53	.82	1.01
Total	..	...	.366	...	..	3.17

For explanation: $I_o = 1.12\ bh^3$; g = distance from c. of g. of small section to neutral axis.
I for half section: $I = \Sigma\ I_o + \Sigma\ ag^2 = .37 + 3.17 = 3.54$ (all dimensions in feet).
I for whole section: $I = 2 \times 3.54 \times 12^4$ (all dimensions in ins.).

TABLE B.—LOADS, MOMENTS AND MAX. TENSILE STRESSES FOR LINE OF FRACTURE SHOWN.

No. of line.	Load. Description.	Weight.	Assumed lever arm in ins.	Moment in 1,000 in lbs.	Max. tensile stress due to Mom. $f = \frac{M\,y}{I\,x}$.
1	Concrete	14,300	45	640	..
2	Earth filling below crown at 100 lbs. per cu. ft.	4,400	27	120	65
3	Earth above crown: 4 ft. at 100 lbs. per cu. ft. + live load = 100 lbs. per sq. ft. Total load = 500 lbs. per sq. ft.	76,500	48	3,680	..
4	Total for 1 + 2 + 3	....	..	4,440	460
5	Earth filling below crown at 110 lbs. per cu. ft	4,800	27	130	
6	Earth above crown: 3 ft. at 110 lbs. per cu. ft. + 15 lbs. snow. Total = 345 as per table of groined arches	52,800	48	2,540	340
7	Total for 1 + 5 + 6 as per table of groined arches	....	..	3,310	..

Data: For Case I. and II. (See also Table I., line 39.)

1. Wt. of 1 pier, 35.3 cu. ft. at 150 = 5,300 lbs.
2. Lever arm of pier (assumed) = 1.33 ft.
3. Wt. of 1 groin unit $15 \times 15 \times \frac{7.3}{12}$ at 150 = 20,500 lbs.
4. Assumed lever arm of groin unit (also of any loading on unit) = 2.17 ft.
5. Resistance to overturning of pier + groin unit = 5,300 × 1.33 + 20,500 × 2.17 = 51,000 ft.-lbs.
6. Weight of filling in hollow of groin, 2.48 yds., at 2,700 = 6,560 lbs.
7. Resistance to overturning afforded by filling in hollow = 6,560 × 2.17 = 14,000 ft.-lbs.
8. Wt. of 1 ft. of filling on a groin unit at 100 lbs. = 22,500 lbs.
9. Resistance to overturn afforded by 1 ft. of filling = 22,500 × 2.17 = 48,000 ft. lbs.
10. Wt. of 3 ft. of filling on 1 groin unit = 67,500 lbs.

Assumptions: (as before):

(a_1) *.85 × 20,500 = 17,400
(b_1) ½ × 6,560 = 3,300
(c_1) 1 × 67,500 = 67,500

88,200 causes thrust of 50,000 lbs.

(a_2) $\frac{17,400}{88,200} \times 50,000 =$ *10,000 lbs. thrust assumed due to wt. of groin unit.

(b_2) $\frac{3,300}{88,200} \times 50,000 =$ 1,900 lbs. thrust assumed due to filling in hollow.

(c_2) $\frac{22,500}{88,200} \times 50,000 =$ 12,800 lbs. thrust assumed due to 100 lbs. sq. ft.

(a_3) 10,000 × 8.75 = 87,500 ft.-lbs. Overturning moment due to groin unit.

(b_3) 1,900 × 8.75 = 16,600 ft.-lbs. Overturning moment due to filling in hollow.

(c_3) 12,800 × 8.75 = 112,000 ft.-lbs. Overturning moment due to 100 lbs. sq. ft.

(d_3) 50,000 × 8.75 = 438,000 ft.-lbs. Overturning moment due to total load.

Case I.—Loading terminated abruptly as in Fig. 9.
51,000 *n* (See Item 5 of "Data" above).
= 600,000 (See Item d_3 of "Assumptions" above).
n = 12, that is, full loading could be placed on arch 180 ft. from edge.

Most of the piers of filters 1 to 46 at Pittsburg were concreted monolithic with the roof, and hence offered further resistance (as long as the roof acted continuously), to the extent that their strength as cantilever beams permitted. At 50 lbs. per sq. in. in allowable tension, each pier would add resistance, *R*, computable as follows:

R × height of springing line = 1/6 × 50 × 22 × 22 × 22.

$R = \frac{88,700}{8.75}$ = 10,000 lbs. This is a considerable aid in resisting a thrust of 50,000 lbs. A few piers cracked horizontally at springing.

The various computations above given show the most favorable conditions possible in that they assume continuity of the roof to the extent necessary to bring the roof load on the edge of the pier instead of the center, at the same time bringing the application of the horizontal thrust to the top of the pier instead of the crown of the arch. For winter conditions such assumptions would not be conservative for reasons that will be evident from the early part of the paper.

Reference has been made to bracing unfinished edges of groined roofs where it is desired to fill up to the edge. High roofs, such as that of the filtered water reservoir at Pittsburg, would be difficult to shore, but the filters are much less so. The most convenient place to shore is opposite each pier, placing the upper end of the brace on the intrados, just above the pier, perpendicular to the intrados, and footing the brace at the base for the next pier. A depression could easily be made for such a brace. Such braces for the Pittsburg filters are about 17 ft. long and the horizontal projection of their length is about 14 ft., making the load $\frac{17}{14} \times 50,000 = 60,000$ lbs. An 8 × 10-in. stick of reasonably good timber with proper bearings at each end is adequate, but blocking at either end with bearing across grain would permit movement and cause cracks to open at the springing line and at the base of the piers. Bracing is certainly more reassuring than a margin of unloaded groins, particularly for work that must stand over winter.

Floors of Filters and Reservoirs.

The common form of floors, and methods of computing their volumes is described in the first part of this article. A tabulation of the floors of the various filters and reservoirs would be quite as pertinent and interesting as that for roofs given in Table I. The writer has had time to compile only a few important cases as given in Table IV. The groined arch floor has been as generally satisfactory as the groined arch roof. Groined arch floors are very easily screeded in diamond-shaped pieces joining under piers, along groin lines and along invert lines. The loads on the earth are comparatively light and the thicknesses which have been merely guessed at, so far as the writer knows, have proved adequate. The Pittsburg filters and reservoirs are mostly on very porous ground and the lower layer of concrete was used in the 1904 design in place of puddle. A standard floor with minimum thickness of 6 ins., as in most other plants, and puddle was the alternative design not used. The 1908 Pittsburg filters contain an innovation in the use of a single flat floor lapping onto reinforced pedestal blocks under piers. The economy in material is evident from the table.

Side Walls and Division Walls.

Side walls of filters and reservoirs exhibit the same boldness of design as do the groined arch roofs. They are generally of the so-called aqueduct section but thinner, particularly at and near the crown, than aqueducts are made. This has an added significance because the side wall is generally concreted separately from the ad-

*This was fixed to give same thrust as determined by line of resistance with dead load alone.

*Adjusted to give same thrust as a line of resistance with dead weight alone.

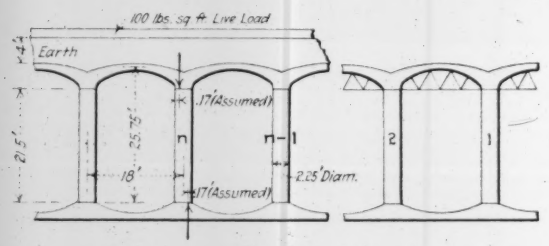


Fig. 9.

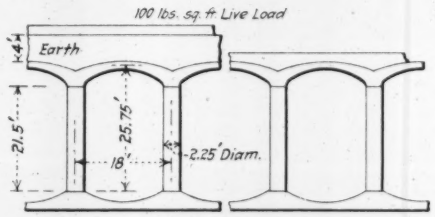


Fig. 10.

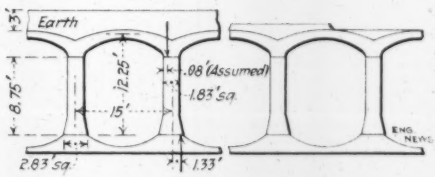


Fig. 11.

FIG. 9 TO 11. VARIOUS METHOD OF LOADING OF GROINED ARCHES DURING CONSTRUCTION.

jacent roof, so as to leave a joint along the crown, whereas the aqueduct is monolithic; and further the side wall has no opposing wall of the same shape and loading to keep a uniform condition, but instead has against it a long stretch of groined roof with varying temperature conditions. At the Washington filtration plant side walls are made like ordinary retaining walls in some cases and the groined arch carried out to them. This gives the same head room near the side walls as near piers or division walls. Dependence must be placed on immobility of filling behind such walls and this dependence appears not to have been misplaced—another example of successful boldness.

Division walls between filters are usually made of the same section as piers, and the floors beneath them are of the same section as the floors beneath piers, though they are sometimes thickened a little.

Comparison of Groined Arch Roof with Reinforced-Concrete Slab and Beam Construction.

(Pittsburg Filtered Water Reservoir Used as Basis.)

Fig. 12 shows a design of a reinforced floor to carry the same load as that assumed for the groin arch of the Pittsburg reservoir, viz.: 400 lbs. of earth and 100 lbs. per sq. ft. live load. The units used for the reinforced concrete are as follows:

Compression in concrete..... 600 lbs. per sq. in.
Tension in steel (mild)..... 16,000 lbs. per sq. in.
Ratio of moduli of elasticity..... 15
Straight line variation in compression of concrete
Moment which beam will carry..... $94\ bd^2$
Per cent. of steel..... 0.67

Beams and slabs designated for moment of $\frac{1}{12}wl^2$ both at center and over supports. Any larger moments due to partial loading assumed to be cared for by increased stresses, since the final conditions are so uniform.

Fig. 12 shows a roughly worked out design for the purpose of comparing costs. No shear computations have been made but rods are bent up "by eye." The design differs from much of the commercial practice in that the compression in the lower parts of the beams at supports where no slab exists has been provided for by deepening the beams instead of by assuming stronger concrete at these inconvenient points. The downward slope of the bottom of the girders toward their supports is perhaps too abrupt. The girders at supports are not quite deep enough to fulfill the formula $94\ bd^2$, but there is an excess of steel due to shearing provisions. The slab steel has not been worked out but the quantity is assumed to be $\frac{4}{3.4}$ of the quantity obtained by taking 0.67% of the volume from the top to the center of steel at the bottom.

From Fig. 12 the average thickness of concrete in the reinforced construction is 9.7 ins. against 8.22 ins. for the groined arch roof of the Pittsburg reservoir; and the steel and other differences are still to be added. A comparison in more detail is as follows:

Groined arch roof:

$\frac{8.22}{12}$ cu. ft. of concrete at $6.00 per yd. (22 cts. per ft.) $0.15
Charge for forms for each sq. ft..... .10

Total relative cost per sq. ft..... $0.25

Reinforced-concrete roof:

.8 cu. ft. of concrete at $6.90 per cu. yd. (25.5 cts. cu. ft..... $0.207
6.78 lbs of steel at 2.75 cts..... 0.187
Charge for forms for each sq. ft. of roof, say.... 0.15

$0.54

The difference in price of concrete is made up of 40 cts. due to using 1:2:4 for the reinforced concrete instead of 1:3:5 as for the groined arch concrete, and 50 cts. assumed extra cost of rehandling or otherwise carefully placing the concrete around the steel. All assumed prices are intended to be so chosen as to favor the reinforced construction, if either. The reinforced construction might be credited with the filling in the hollow of the vaulting not necessary with the flat roof. This would amount to $\frac{4.34}{324}$ = .013 yds. of earth filling per sq. ft., but this would cost less than ½ ct. In this comparison it is assumed that the high water line could be at the same distance below the flat roof as below the arched roof and that air pockets formed above high water by the beams are unobjectionable.

It may be objected that it is not fair to compare a reinforced-concrete structure capable of bearing, with a certain factor of safety nearly 2 on the elastic limit, a load per square foot of 400 lbs. of filling and 100 lbs. live, with a structure that, whatever it is thought capable of *is* carrying only perhaps 315 lbs. per sq. ft. in addition to its own weight and doing this with an unknown margin of safety. A sufficient close correction to the quantities in the reinforced construction resulting from reducing the load to 315 lbs. can be arrived at by assuming that the "effective" concrete and all the steel varies as the equare root of the total load, thus:

Loads as used in design — Dead load 120 lbs. per sq. ft.
Live load 500 lbs. per sq. ft.
620

Revised loads — Dead load say 105 lbs. per sq. ft.
Live load 315 lbs. per sq. ft.
420

Effective concrete say 8/10 of total.
Then revised effective concrete =

$$\sqrt{\frac{420}{620}} \times 8/10 \times .81 = .53 \text{ cu. ft. per sq. ft.}$$

Total revised effective concrete =
$53 + \frac{1}{2} \times .81$ = .70 cu. ft. per sq. ft.

TABLE II.—COMPUTATION OF SAFE LOADING ON ROOF OF FILTERED WATER RESERVOIR, PITTSBURG, ADJACENT TO AN UNFINISHED EDGE OF ROOF. CASE II. FILLING TAPERED AS IN FIG. 10.

No. of pier and groin (1)	*Resistance to overturning, 1,000 ft.-lbs. (2)		Total resistance to overturning, 1,000 ft.-lbs. (3)	Safe Loading on Next Groin Unit. (For thrusts, see "Assumptions" above listed.) (4)
1 to 5 inc.	Each	76	380	Own weight (A_3) 310
6		76	456	Own weight 310 Filling in hollow (b_3) 69 379 456 − 379 = 77 ⌭ $\frac{77}{\dagger 375}$ = .20 fill over crown.
7	Own weight 76 Fill in hollow .20 × 62 = 22 12 110		566	566 − 379 = 187 ⌭ $\frac{187}{375}$ = .50 fill over crown.
8	.75 × 22 = 98 .50 × 62 = 31 129		695	695 − 379 = 316 ⌭ .84 fill over crown.
9	.84 × 62 = 98 52 150		845	845 − 379 = 466 ⌭ 1.24 fill over crown.
10	1.24 × 62 = 98 77 175		1,020	1,020 − 379 = 641 ⌭ 1.71 fill over crown.
11	1.71 × 62 = 98 106 204		1,224	1,224 − 379 = 845 ⌭ 2.25 fill over crown.
12	2.25 × 62 = 98 140 238		1,462	1,462 − 379 = 1,083 ⌭ 2.90 fill over crown.
13	2.90 × 62 = 98 140 278		1,740	1,740 − 379 = 1,361 ⌭ 3.65 fill over crown.
14	3.65 × 62 = 98 225 323		2,063	2,063 − 379 = 1,684 ⌭ 4.00 fill over crown. + 50 lbs. per sq. ft. live load.
15	4.50 × 62 = 98 280 378		2,441	2,441 − 379 = 2,062 ⌭ 4.00 fill over crown. + 150 lbs. per sq. ft. live load

*See Items 5, 7 and 9 of Data above. †See Item C_3 above.

TABLE III.—COMPUTATION OF SAFE LOADING ON ROOF OF FILTERS 1 TO 46 AT PITTSBURG, PA., ADJACENT TO AN UNFINISHED EDGE OF ROOF. CASE II. FILLING TAPERED AS IN FIG. 11.

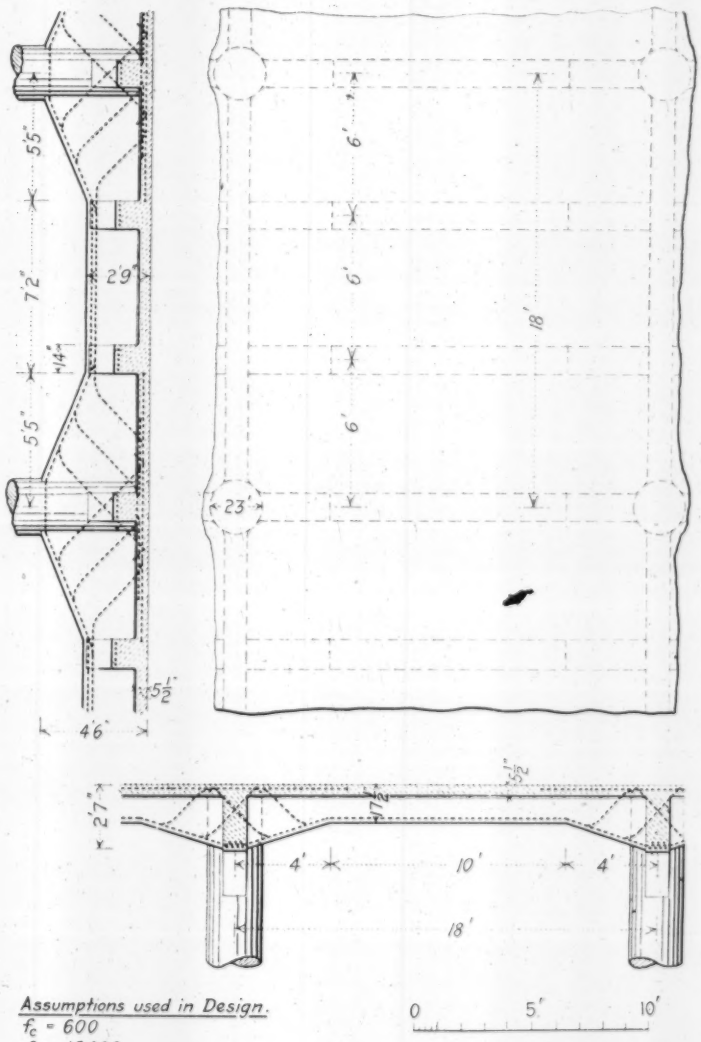
No. of pier and groin (1)	*Resistance to overturn, 1,000 ft.-lbs. (2)		Total resist. to overturn, 1,000 ft.-lbs. (3)	Safe Loading for Next Groin Unit. (For thrusts see Assumptions above listed.) (4)
1 to 3 inc.	Each	51	153	Own weight (a_3) 87.5 ft.-lbs. Filling in hollow (b_3) 16.5 ft.-lbs. 104 ft.-lbs. 153 − 104 = 49 ⌭ $\frac{49}{\dagger 112}$ = .44′ fill above crown.
4	Own weight 51 Fill in hollow .44 × 48 = 21 86		239	239 − 104 = 135 ⌭ $\frac{135}{112}$ = 1.20′ fill above crown.
5	51 + 14 = 65 1.2 × 48 = 58 123		362	362 − 104 = 258 ⌭ $\frac{258}{112}$ = 2.30′ fill above crown.
6	2.3 × 48 = 65 110 175		537	537 − 104 = 433 ⌭ $\frac{433}{112}$ = 3.9′ fill above crown.

*See Items 5, 7 and 9 of "Data" above.
†See Items (C_3) above.

TABLE IV.—DATA ON FLOORS OF A FEW FILTERS AND RESERVOIRS.

*Reference No.	Date.	Location.	Engineer.	Pier spacing, ft. ins.	Size of base on which pier rests, ins.	Thickness under pier, ins.	Thickness at invert, ins.	Average thickness, ins.	Thickness of puddle, ins.	Kind of soil.	
6	1899	Albany, N. Y.	Allen Hazen.	13 8	30 × 30	15	6	8.2	0	.....	Filters.
9	1901	Phila, Pa, Low. Roxbor'gh	John W. Hill.	15 10	36 × 36	14	6	8.0	12	.....	Filters & reservoir.
11	1902	do., Upper Roxborough.	"	15 10	36 × 36	14	6	8.0	12	.....	Filters & reservoir.
18	1903	Philadelphia, Pa., Belmont.	"	15 3	36 × 36	14	6	8.1	12	.....	Filters & reservoir.
19	1903	Philadelphia, Pa., Belmont.	"	15 10	24 × 24	14	6	7.7	12	.....	Filtered water res.
20	1903	Phila., Pa., Torresdale.	"	15 10	36 × 36	14	6	8.0	12	.....	33 filters.
21	1903	Phila., Pa., Torresdale.	"	15 0	36 × 36	14	6	8.1	12	.....	22 filters.
22	1903	Phila., Pa., Torresdale.	"	15 10	24 × 24	14	6	7.7	12	.....	Filtered water res.
23	1903	Washington, D. C.	Col. A. M. Miller.	14 0	32 × 32	15	6	8.2	0	.....	Filters 1-24.
30	1903	†N. Y. City, Stormville.	Wm. B. Fuller. 17 6 × 16	8	36 × 36	16	8	§10.0	0	Rock, hardpan, etc.	200 filters
31	1903	†New York City, Hillview.	" 22	4	30 × 30	24	12	§14.5	0	xRock.	Filtered water res.
39	1904	Pittsburg, Pa.	Morris Knowles.	15 0	36 × 36	17	8	§10.4	0	Mostly silt.	Filters 1-46.
40	1904	Pittsburg, Pa.	"	18 0	27 × 27	22	10	§12.6	0	Sandy gravel.	Filtered water res.
41	1908	Pittsburg, Pa.	"	14 10	———	φ	———	6.6	0	Sandy gravel.	Filters 47-56
44	1909	‡New York City, Hillview.	J. Waldo Smith.	22 0	36 × 36	‖18	6	‖8.0	0	Hardpan.	Filtered water res.

*Same as in Table I.
†Not built.
‡To be built without cover and with flat floor—above design abandoned.
§Two layers of concrete instead of 1 layer of concrete and puddle.
‖Bottom of floor under pier is 6 ins. higher than bottom under invert, thus saving a pyramid of excav. and concrete 22 × 22 ft. × 6 ins. high, or an average of 2 ft. over whole area. Excavation somewhat complicated. Done to improve drainage of excavation under piers.
φPier foundation 6 × 6 ft. × 12 ins. thick, reinforced with 16 ¾-in. square steel bars. (Total weight = 126 lbs.). Rest of floor 5 ins. thick, except for 4 × 12-in. water stops under diagonal joints.
xDesign for Rock-Rock now known not to exist.



Assumptions used in Design.
f_c = 600
f_s = 16,000
n = 15
% of Steel = 0.67 %
K = 94 in Formula M = Kbd²
M at Center of Span = $\frac{1}{12}wl^2$
M over Supports = $\frac{1}{12}wl^2$

Approximate Quantities.
Concrete = .81 Cu. Ft. per Sq. Ft. (Average)
Steel = 6.78 lbs. " " "
Slab Reinforcement not shown.

FIG. 12. APPROXIMATE DESIGN FOR REINFORCED-CONCRETE BEAM AND SLAB CONSTRUCTION TO CARRY LOAD OF 500-LBS. PER SQ. FT. IN ADDITION TO DEAD LOAD.

(Made for Economic Comparison with Plain Concrete Groined Arch Construction.)

Revised steel quantity =

$$\sqrt{\frac{420}{620}} \times 6.78 = 5.6 \text{ lbs. per sq. ft.}$$

Revised relative cost = .70 × 25.5¢ + 5.6 × 2.75¢ + say $0.14 = $0.47 per sq. ft.

Again it may be said that it is not necessary to space the piers so far apart. The cost of the effective concrete and the steel would be reduced approximately in direct ratio of the revised to the original span. But the saving computed by reducing the span would be more apparent than real since the high piers are already about as slender as desirable and the cost of forms per square foot of roof would probably increase with the greater number of units to be erected to produce the given area of roof. One who has watched the extremely rapid work of placing the groined roof by cableway (an average of 200 cu. yds. per day has been placed with one cableway during several months) would probably never feel moved to substitute, at least on a large plant, the complicated reinforced construction, even if the factor of safety of the groined arch is a little obscure. The New Haven filter plant is an example of the use of reinforced concrete for a small plant.

In conclusion the writer wishes to make acknowledgment to Mr. Morris Knowles, Chief Engineer of the Bureau of Filtration, Pittsburg, Pa., for permission to use the Pittsburg work as a text and for making it possible to insert the plate* from the progress photographs taken during the construction of the Pittsburg plant by the T. A. Gillespie Co.

A HEAVY PIVOT-BEARING DRAWBRIDGE, having its entire weight carried by the pivot when swinging, is to be built by the Paris, Lyons & Mediterranean Ry. (France) on a cut-off for the improvement of its main line between Paris and Marseilles. The line extends from Miramas to Estaque, and crosses the Caronate basin by a bridge having eight fixed deck spans of 270½ ft., two deck spans of 168 ft., and an equal-arm through-truss swing span of 370 ft. This span is said to be the heaviest swing bridge ever built in which the entire weight (when swinging) is carried by a pivot bearing. The total weight will be about 1,350 tons, and the load on the bearing will be about 3,557 lbs. per sq. in. The pivot is designed to consist of a lenticular disk of phosphor-bronze 33¼ ins. diameter, fitted between two plates of annealed chrome steel having concave bearing surfaces to fit the bronze disk. The swing bridge crosses the head of an existing canal, and provides 141-ft. clear opening for the projected canal from Marseilles to the Rhone, as well as for a future military canal. The clear headway is 75½ ft., permitting the passage of fishing boats and other vessels which ordinarily use the canal, so that it is only occasionally that the bridge will have to be opened. Two electric motors of 65 HP. are provided (one for reserve), and the operating machinery is mounted in a house between the trusses at the middle of the bridge.

*Not published in this reprint.

A New Separator for the Removal of Slate from Coal.*

By V. H. WILSON.

The chief objects sought in designing the separator to be described were the avoidance of all impacts and the consequent wasteful chipping of the coal, the doing away as far as possible with hand picking, the removing of flat slate without wasting the flat coal, the exposing of the operation to view so that the exact action of the machine could be observed at all times at a glance, and finally the possibilities of adjusting the machine while in operation.

A traveling separating belt, mounted on two shafts, is made to move upwardly on its upper run by means of the drive pulley. The belt, made of metal slats attached to a specially designed link belt, is inclined forwardly as well as transversely, and at such angles as are suitable for the proper separation of the material to be treated. Its transverse inclination is adjustable, while running, by means of a lever and ratchet wheel.

The material is fed in a continuous stream upon a pan at the farther and higher end of the machine through a properly constructed feed chute. As it slides off the pan, the upwardly moving belt immediately spreads the material out into a well-spaced stream, so that the coal may slide down against the guide and off at the lower right-hand corner of the machine, while the slate, adhering to the belt, is carried upwardly to the left-hand side and thus out of the moving stream of coal.

The coal is conveyed away from the machine in any desired direction by a suitable chute, and the slate in like manner is collected along the slate apron at the left in a similar chute and disposed of as desired.

The capacity of the machine, which varies somewhat with the nature of the material, but chiefly with the size of the coal, ranges from five tons per hour on pea size to 25 tons per hour on steamboat size. The operation is distinctly open to view.

The saving by preventing the loss in chippings, which loss ranges in other separators from 5% to 20% of the coal treated, is enormous. The chippings on this new separator amount, on an average, to less than 0.5%. On the low basis of 5% saved, the amount would be $18,750 on every 100,000 tons of prepared sizes shipped. One installation of ten machines, now in operation for more than a year, shows a gain of 10.5% in the prepared sizes, with an output of 11,500 tons per month, or a gain of $4.52 per month. In addition to the gain by the prevention of chippings, a saving in labor of $500 per month has been affected.

It is not possible in all cases to do away with hand-picking by the use of the separator because of the occasional presence of pieces of slate having coal faces and pieces of coal having slate faces.

The equipment pays for itself in from 10 to 80 days. In treating such material as has been described in connection with the jig, it has been found most effective to use a three-stage process. The first stage has a set of three separators, turning direct to the pocket, without appreciable loss from chippings, 60% to 75% of the total coal in the material, which includes all the glassy fractured cubical coal and all of the flat coal not having slate faces. The second stage has likewise a set of three separators so adjusted as to remove all the flat slate and all the pure slate and rock. With the foregoing mixture or forms trimmed, so to speak, in this way, we have left flat coal with slate faces. Passing this product, which contains only from 25% to 40% of the total coal, to a jig as the third process, a very satisfactory separation can be made.

By this three-stage process the losses have been reduced to a minimum, or, to put it the other way, the gains as shown in the installation of the ten machines previously mentioned have been as high as 19.5% in the prepared sizes.

*Abstract of an article in "The Yale Scientific Monthly" for March, 1910.

The Drainage and Protection of Lands Subject to Overflow.*

By CHARLES W. BROWN.†

Bordering our main waterways lie large areas of rich soil, formed by silt deposits from tributary streams. Such lands require for their agricultural development both drainage and protection, but they must be in large tracts to make the scheme feasible. In other words, the ratio of length of levee to area of land must be small or the cost will be too great a burden. This is true especially in a closed district where gravity discharge cannot be maintained.

The topography of the entire drainage area should be carefully plotted, and the run-off of high lands computed for gathering in open channels large enough for the storage of sudden dashes as well as for carrying ordinary flow. The drainage may be provided for both into open channels and tile or covered drains. The uncertainty of gravity drainage makes mechanical means at an increased cost preferable in most cases, and this will usually influence the general scheme by increasing the size of open channels.

The run-off from high lands should be diverted, if possible, and carried without the drainage area. If not, the water should be collected in channels or basins so that the maximum rate of pumping could hold the elevation of the water below danger stage. Factors to be considered in main drains are: run-off, seepage and storage, thus increasing the size of usual channels. Covered drains afford little or no storage and should be limited to suit mains, in areas not affected by hill water.

The annual rainfall in this state approximates 40 ins. That of May or June frequently reaches approximately 5 ins. Therefore, ¼-in. in 24 hours is sufficient allowance for removal in well-drained soils. The bottom land being nearly level and generally covered by some crop or cover, the run-off is thereby retarded, but the run-off from higher lands may sweep over the bottom lands unless intercepted by open channels.

The seepage varies with every locality. In clay subsoil the flow is very light and is most apparent in the joint clay, or the strata of mixed loam and clay from 2½ to 3 ft. below the surface. At about this point ditches usually begin to slough off and scour. In sand or gravel sub-soil, the seepage may amount to as much as 100 gals. per 24 hours per sq. ft. of exposed face, and when under pressure severe boils occur, not only in ditches but over the entire area affected by such subsoil. The amount reaching the channels from the drains will depend on the depth of strata between the tile and gravel beds, and this is the factor hardest to be determined and the most important to the success of any project.

The storage of hill water not diverted, or from high lands which come down rapidly, must be added also, giving a ditch for the combined purposes from 40 to 100% larger than for ordinary flow requirements.

In many of the closed districts much of the hill water may be diverted and discharged directly into the main stream without going through the district. At the point of diversion the creek bottom is reckoned from the flood elevations in both main and tributary stream, and the protecting levee graded accordingly, as the flood discharge from a rapid creek sweeps down in a wave in some conditions and meeting the flood waters of the main stream overtops levees otherwise of safe height. Diverting channels are difficult to maintain unless provided with settling basins or reaches of large area where the creek flood may spread over a large area and spend its velocity and deposit the silt in suspension. Channels through such basins are apt to change, but this affects the dry-weather flow only, which is of small moment. The velocity should be not more than 0.3 to 0.4 ft. per sec. through these basins.

Without such provision for sedimentation the flood waters are apt to gorge, and complete filling of the channel occur, with consequent destruction of the adjacent levees. The writer recalls a channel about one mile in length completely filled in this manner. The velocity in the drainage channels should be low, and to prevent scouring should be as low as consistent with economy in construction, and compensated by increased width to supply the maximum pumping capacity. It has been demonstrated in many instances that the flow in main channels calculated to carry the entire run-off is not sufficient to supply the pumps without unduly increasing the depth of water in the channels. The friction from vegetable growth and irregular shape of sides and bottom frequently reduces the volume by nearly half the estimated rate.

The sub-mains of tile should be designed of sufficient capacity for seepage and rainfall, and placed as low as possible. The depth will depend on the sub-soil, but they should never penetrate the sand and gravel unless the bottom of the trench is well sealed with gumbo or socket tile, well cemented. It is better to make the drain shallower than to long continue in sand and gravel, as the object of these drains is to reduce the elevation of permanent ground water, thereby increasing the storage and the time for handling the flow. The laterals should give, if possible, 3½ to 4 ft. of well-drained soil for storage, and reduce the ground water permanently to that depth.

Submerged lands are usually somewhat alkaline and the alkalinity increases with the rise of the ground water. If drains are ample and deep, the flow is from the surface to the drain and the alkali is carried off, improving the land for agriculture. The size of tile drains I estimate from the formula for mean annual rainfall, which is slightly less than ¼-in. per day. For tracts that are well tiled this is ample, but for land that is tiled as farm tiling is usually done (by laying the drains in swales only) no formula is adequate. Conditions of surface flow in bottom lands render tiling more effective, and at the same time exact areas may be drained since waters of other areas seldom affect them.

Laying tile along contours for intercepting seepage and hill water is unsatisfactory, if improperly done; as the head is greater across the contour than with it, therefore the flow leaps over. But by connecting the tile laid with the contour into branches laid across the contour and taking the flow at frequent intervals, the results are good. Sufficient open channels should be constructed for the storage and carrying of large volumes of water, and so placed that natural conditions may be taken advantage of as regards minimum excavations and reaching low tracts.

In open channel construction where the soil is firm with a clay bottom, the channel may be cut for the requirements of flow and storage. But where the soil is sandy or with a sandy or gravel sub-soil, the channel should be cut wider and deeper than the above requirements, and allowed to fill by the surface soil sliding into the channel, and sealing over the bottom to a depth of 1 or 2 ft. to prevent too rapid flow of the ground water in the channel. The portion of the main channel where the waters of the entire district unite should be of such depth and width that the lowest possible head will provide a flow of at least 50% in excess of the pumping capacity.

Some investigations are being made by the writer as to the effect of the rise and fall in the river and on the flow in the sub-soil from which we hope to obtain a constant showing the permeability of the soil and the velocity of the flow.

In the construction of open channels it has been customary to specify a fixed width of berm, without regard to the soil conditions. The width is usually fixed by the machinery and apparatus to be used, thus basing the execution of the work upon the limiting conditions rather than on the requirements of the work. The width should be varied with the changing character of the soil, and proportioned to the angle or the slope which the soil will naturally take, with due regard to the fact that wet banks stand better than those that are alternately wet or dry. The sloughing off of banks has been frequently stated as having the effect of filling the corners of the ditch next the bank, but it has been my experience that when this occurs, the material extends almost across the ditch, so that the width of the channel is not much affected, but the depth is decreased. Excessive spoil deposited on the berm has the effect of increasing the height and decreasing the stability of the banks, and in effect is just the reverse from like conditions in levee construction.

In channel construction with the dipper dredge, the width of the channel usually requires that the hull be secured by anchors or side spuds. These have a detrimental effect by breaking down the berm or crushing the same so that the earth affected soon slides into the channel. This will be the case with equipment of this character wherever the machine follows up the cutting, but where the machine precedes the cutting, these effects are not noticeable. When a dipper dredge is used on a small channel, a small dipper with extra long boom should be used to deposit the spoil as far from the channel as possible, and to strain the banks by the anchors as little as may be. It is economy in construction to pay a higher price per yard for work done in this manner than for work done by apparatus which leaves the work in less durable condition.

For estimating the flow it has been my practice to use as a basis 40 ins. of rainfall annually, with half to be drained off and half of this as a mean monthly flow. From this I plot a curve showing the monthly flow expressed as a ratio of the mean monthly flow. For instance, the flow in April and August is respectively 1.64 and 0.23 of the mean monthly flow. The seepage is expressed as a ratio of the mean monthly flow, which we find in April to be the greatest and equaling 1.38 of the mean monthly flow. The two factors are combined in a ratio of pumpage which is equal to 2.26 of the mean monthly flow. During 17 years of observation, March, April, May and June have been the months which required the greatest pumping, and the aggregate was practically half the entire flow for the year in districts which require pumping.

In a district now under investigation, the mean annual rainfall which we estimate would require pumping to eliminate, totals 508,200,000 cu. ft., but with the addition of seepage we estimate that 66,183,500 cu. ft. must be pumped. The capacity of the pumping equipment being based on the latter requirement, shows a capacity of 30,000 galls. per minute.

The design of the pumping plant will be determined by the economy required, and this can be determined only by figuring the maintenance of a plant of high economy with one of less economy and determining the initial cost. In districts which require pumping for from four to six months annually, the equipment which shows the lesser mechanical efficiency will in most cases show the higher commercial efficiency. But the district which requires from eight to eleven months' pumping, a plant of high mechanical efficiency will also show the best commercial efficiency.

In the protection work, creeks that must be diverted have to be taken into account as before stated, and the levee construction should be based on an elevation from 2 to 5 ft. above the highest recorded stage of water. This applies both to the main stream and the tributary streams.

The width of crest, the inner and outer slope and width of berm should be determined by the character of soil with which the levee is to be constructed. In heavy black soil, slopes of 1 on 2½ on the outer face and 1 on 1½ on the inner face, stand well where not subject to wave action. A mixture of sand and clay should have a slope of 1 on 4 or 5 on the outer face and 1 on 2 on the inner face. If the levee exceeds 10 or 12 ft. in height, it may be preferable to construct on the inside of the levee (and at mid-height) a berm of 6 to 8 ft. A levee constructed entirely of bank sand, and submerged within 1 ft. of the crest, took a slope under water of 1 on 4, and successfully withstood the flood at that height.

The soil under the base of the levee should be thoroughly plowed. If it is a firm, close soil, by plowing the base and throwing the alternate furrows so as to form a series of dead furrows, the construction of muck ditch may be dispensed with. But if the soil is light and of pervious nature, the muck ditch becomes necessary and a depth of from 3 to 4 ft. with 1 to 1 slopes, is sufficient. In porous and sandy soil the muck ditch should be cut to the greatest depth permissible with the character of the soil. The base should also be free of all brush and rubbish, stumps blown out and grubbed and the holes filled with earth compacted by tamping.

The width of berm in most cases has been limited by the apparatus for doing the work, or by the cost of the right of way, but should be determined by the slope which the soil will take when submerged. Borrow-pits should be required which will give a slope to their banks not less than 1 on 2, or a slope which the action of the water will give to them. When constructed with teams and scrapers, the width of berm and depth of borrow-pit are easily limited and can be made to conform to the requirement of the soil, but when constructed by such machinery as a dipper dredge the width of berm is usually much less than it should be, being limited by the ability of the dredge to deposit the material the required distance.

In levee construction, depositing material on the berm is not a detriment as in the case of ditch construction, as loading the berm while it would have the effect of causing the same to slide into the borrow-pit, would not be so detrimental, and the material so deposited serves to resist the wave action and erosion by the current. Wave action is the most detrimental of all factors in levee maintenance, therefore a foreshore covered with brush and timber should be left or provided. But brush or undergrowth should not be allowed on the levee or berm, as this furnishes cover for burrowing animals and the roots loosen the soil by wind action, as well as tending to mellow it. Among other things to be avoided are roadways on the crest of the levee and railway fills constructed on old trestle work. The roadway will lead to dangerous washes and the fills on trestle work to seepage that is difficult to control.

Finally, the improvement of such tracts must fall within a cost per acre of from $20 to $30 if the land owners are to be induced to unite on the proposition.

*Abstract of a paper read at the annual meeting of the Illinois Society of Engineers and Surveyors, at Cairo, Ill., Jan. 26, 28.

†Civil Engineer, Jacksonville, Ill.

THE GROSS REVENUE OF THE BELL TELEPHONE System for 1909 was $150,000,000, in round numbers. This does not include the revenue of the independent telephone companies. Of this total revenue, about $45,000,000 was devoted to current repairs, maintenance and depreciation, and $24,000,000 was paid in dividends. There are now over 5,000,000 telephone stations in the Bell system, over 10,000,000 miles of wire, and the traffic has increased to nearly 20 million connections a day, or 6½ billion connections a year. The additions to Bell telephone plant during 1909 amounted to over $28,000,000.

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Mr. James J. Hill, in a public interview last week, expressed the opinion that another period of railway congestion is in sight and that the railways ought to spend about $1,600,000,000 a year for the next five years in the enlargement and improvement of terminals and in increased trackage, in order to meet the demands made upon them. Commenting on the above Mr. W. C. Brown, president of the New York Central Lines, declared that there was greater need of business for existing railway facilities than of additional terminals and that at the Eastern seaports at least, the export traffic in grain has now disappeared and present railway terminal facilities are amply sufficient to care for the business.

In our opinion, Mr. Brown is nearer right than Mr. Hill. Probably the coming fall will see a season of car shortage and congestion of traffic; but we are inclined to the belief that the mere lavishing of money on additional rolling stock or on enlargements of yards located on expensive city land, is not the best cure for either of these conditions. Better methods of distributing and moving cars can accomplish more to relieve car shortage, than the investment of millions in new cars. Better methods of operating terminal yards and of handling freight can effect more in relieving congestion than mere additions to area.

An extended agitation for higher wages has been going on for some months among railway employees of all classes, both east and west. At one time there appeared to be some danger of a general strike and suspension of railway traffic. This disastrous event now appears to have been averted by the granting of a horizontal increase in railway wages that will be practically universal in extent.

During the past week several of the larger railway systems have finally concluded negotiations resulting in a substantial increase of employees' wages. Among the most notable of these is the Pennsylvania System, which has announced an advance of 6% in the wages of all permanent employees of the company receiving less than $300 per month. It is stated that this is the third general increase in wages that has taken place on the Pennsylvania System in the last eight years. In December 1902 all employees receiving less than $200 per month were increased 10%. In December, 1906, another increase of 10% was made. The present increase of 6% means that the wages paid when this advance takes effect will be 28.26% more than the wages paid in December, 1902. It is stated, in addition that other increases for various classes of employees have been granted during this eight years, so that the total cost of labor to the Pennsylvania System for employees in a given grade will average fully 33% greater after this date than in 1902.

While an increase in wages has been forced upon the railway companies by the increase in the cost of living, the railways' expenses for other things than labor have increased in even greater proportion. The only way in which it has been possible for the railways to meet these large additions to expenditure without increasing their charges per passenger mile and per ton mile has been the reduction in the cost of transportation effected by improved methods of operating and increased volume of traffic.

The influence of this widespread advance in wages upon the labor market is of great importance. On the Pennsylvania system alone the railway lines affected by the horizontal increase just granted paid to their employees in 1909 $131,823,937, and the total number of employees affected was just short of 190,000.

Ever since the development of steel manufacture on a large scale, the tendency toward continuous working for seven days in the week has been very strong. The injurious effect of this upon the labor employed has been generally admitted by all careful and thoughtful observers. A radical step has now been taken by the chief officers of the United States Steel Corporation to cut down the amount of Sunday work. Two years ago, on April 23, 1907, a resolution was adopted by the Finance Committee of the Board of Directors recommending to the subsidiary companies "that all work be suspended on Sunday at all steel works, rolling mills, shops, quarries and docks; that there shall be no construction work, loading or unloading of materials." The sole exception made was such repair work as could not be done while the mills were in operation, and such departments as required continuous operation, such as the blast furnaces.

On March 21 a telegram was sent out to the subsidiary companies from Chairman Gary, President Corey and Vice-President Dickson ordering strict compliance with the resolution of two years ago. The telegram says:

> All of us expect and insist that hereafter the spirit of the resolution will be observed and carried into effect. There should and must be no unnecessary deviation without first taking up the question with our Finance Committee and asking for a change of the views of the Committee, which probably will not under any circumstances be secured.

The heads of the Steel Corporation deserve much credit for this radical step. The sacrifice of life and health and morals of employees due to the strain of continuous work, as revealed by the investigations of the Pittsburg "Survey," revealed the necessity for radical action. With the enormous output obtainable from a plant by modern machinery and methods, it surely is not necessary to sacrifice the employees' welfare in the way that has so long been followed. We trust the example of the Steel Corporation will have influence with other steel producers in bringing about better conditions for workers in the steel industry everywhere.

A number of accidents to passengers occur at subway stations in New York City each week as a result of the gaps between the edges of car platforms and station platforms. These gaps necessarily exist at curved station platforms, and in some cases amount to ten or twelve inches—quite enough to admit the foot of an unwary passenger, especially in the case of a woman or child. In the dense crowds, which at certain hours of the day enter and leave the subway cars almost in lockstep, it is next to impossible to obey the guard's shouted warning to "Watch your step!"

The great majority of such accidents are not attended by any more serious results than the abrasion, bruising and considerable discomfort of the victims. Nevertheless it is ridiculous that an inconvenience so real and at the same time so easily to be remedied by the addition of extensible edge boards to the car platforms should continue. Such boards could be arranged to be moved out when the car doors open and should not prove difficult to design nor expensive to install.

In the new subway constructions contemplated in New York City the difficulty is to be corrected by locating all station platforms on tangents, but it was not foreseen in the case of the original subway of the Interborough Rapid Transit Co., and the expense of relocating the present platforms would, of course, be prohibitive. The matter of extensible car platforms has already been considered by the Public Service Commission, and the practicability of extensible edges for the station platforms was also investigated. But no action has as yet been taken toward securing relief.

A correspondent calls our attention to an evident error in an editorial in our issue of March 24, p. 343, referring to the Raldiris bill introduced in the New York legislature. We said:

> This bill places the control of engineering practice in New York State in the hands of the Board of Regents of New York University. This is a body which in our opinion is now exercising a most pernicious influence through its authority over the educational system of New York State.

Our correspondent reminds us that the Board of Regents which has autocratic powers over the educational system of New York State is the "Board of Regents of the University of the State of New York," an entirely different institution from New York University, which is an educational institution located in New York City and famous in recent years because of its possession of a "Hall of Fame."

Perhaps a word of explanation is due also as to why we believe the Regents' control over New York's public school system to be pernicious. We are well aware that the Regents themselves are men of high standing in the community and their intentions towards the school system are doubtless most praiseworthy. Their aim is, without doubt, "to maintain a high standard" in the school work; but the unfortunate result is that the courses of study and the methods of instruction are framed toward the end of "passing the Regents examinations" rather than giving the children in the common schools a practical education for the duties of life. We forbear pursuing the subject further but deemed it best to make this explanation lest our remarks be taken as unduly reflecting upon the gentlemen composing the Board of Regents.

The Bill to Increase the Corps of Engineers, U. S. A.

In our issue of Jan. 27, we gave our hearty approval to the Bill for increasing the Corps of Engineers of the Army, believing as we then did, that this bill offered a first step toward proper recognition of the U. S. Assistant Engineers, and for the admission of engineers graduating from other schools than West Point to the Corps.

This bill [H. R. 7117] was favorably reported to the House from the Committee on Military Affairs on March 23; but it has suffered such amendment in the committee that civilian engineers can no longer be expected to take an interest in its passage.

As originally drafted the bill read that "vacancies in the grade of second-lieutenant shall be filled as far as practicable by promotions from the cadets at the United States Military Academy, *selected as heretofore*." This section of the bill has been struck out and rewritten, and the last three words printed in italics above are omitted. The bill now reads that "vacancies in the grade of second-lieutenant in the Corps of Engineers shall hereafter be filled, as far as may be consistent with the interests of the

Military Service, by promotions from the Corps of Cadets at the United States Military Academy." There is also a requirement that the proportion of any West Point graduating class commissioned as Engineers must be at least as great as the proportion between the total engineer officers authorized by law and the authorized officers in all other branches of the Army.

Under such a law the opportunities for graduates of other engineering schools than West Point to obtain a commission as a Second-Lieutenant of Engineers are likely to be extremely few.

As for the chances for the experienced Assistant Engineers now in the Service the only reference to them is the following:

> That whenever it shall be necessary, in order to properly prosecute works of river and harbor improvement, the Chief of Engineers is authorized to detail for duty in charge of river and harbor districts, or as members of boards of engineers any assistant engineers in the employ of the Engineer Bureau of the War Department.

There is no provision here as to the proper compensation for civilian engineers placed in such responsible positions. Doubtless the old-time precedent will be followed that the civilian assistant, with no tenure of position whatever, is obliged to accept smaller salary than the officer over him, who has a life position with retirement allowance on reaching the age limit.

As our readers know, Engineering News has always held a conservative position with respect to the long-standing controversy as to the position of the civilian engineers engaged in river and harbor work. We have opposed as unwise the various efforts put forward from time to time to make such radical changes as the placing of this work entirely in the hands of a civilian bureau, and we have been as frank in our appreciation of the splendid work done by the Corps of Engineers as we have been explicit in defending the claims of the civilian engineers for a square deal.

We know, moreover, from many letters received that our position has the hearty approval of the best and broadest men among the officers of the Corps and among their civilian assistants.

We, therefore, feel free to say that the amendments adopted by the House Committee, and the rejection of other amendments which would have given some measure of recognition to the civilian engineers deserves most hearty condemnation by the civil engineering profession.

We say most hearty condemnation, because the bill as it now stands is a farce. It falls far short of doing justice to the civilian engineers. It ignores the matters of public interest which are at stake. The bill is entitled "A Bill to Increase the Efficiency of the Engineer Corps," but it is only too evident that those who have tinkered the bill in committee have been much more concerned in retaining a West Point monopoly on every task than in promoting real efficiency.

On one point at least we are certainly entitled to protest. The committee supports the bill in a report [No. 812] in which they quote from various authorities and print some extracts from the editorial in our issue of Jan. 27 last. These extracts are so selected as to include what we said commendatory of the engineer officers but each and every statement as to the just claims of their civilian assistants is carefully omitted.

It is a very poor cause which has to use such methods in its defense.

We suggest that engineers or engineering societies desiring to inform themselves on this matter and to appeal for proper recognition of the profession, obtain copies of the bill and the committee report thereon. We may add that the House Committee on Military Affairs (to which the bill ought to be recommitted for amendment along proper lines) is made up as follows: John A. T. Hull, Iowa; Adin B. Capron, Rhode Island; George W. Prince, Illinois; H. Olin Young, Michigan; Julius Kahn, California; James F. Burke, Pennsylvania; Thomas W. Bradley, New York; Frederick C. Stevens, Minnesota; Daniel R. Anthony, Jr., Kansas; Charles L. Knapp, New York; Charles H. Morgan, Missouri; John Q. Tilson, Connecticut; James Wickersham, Alaska; William Sulzer, New York; James Hay, Virginia; James L. Slayden, Texas; Isaac R. Sherwood, Ohio; George W. Gordon, Tennessee; Ben Cravens, Arkansas; S. H. Dent, Jr., Alabama.

The Growing Use of Hypochlorites of Sodium and Calcium as Water Disinfectants.

In view of the growing use and apparent notable success of hypochlorites of calcium and sodium as water disinfectants, and of the half dozen articles on such disinfecting plants printed in our present issue, it seems pertinent to present some facts and comments regarding the history of the subject.

Fifteen years ago an attempt was made to exploit the use of hypochlorite of sodium for the purification of water and sewage and as a universal disinfectant. A small plant was installed for treating the sewage of a few buildings at Brewsters, N. Y., in order to protect the water-supply of New York City, and in addition there was put in at the same place and for the same general purpose a plant for introducing this agent into the East Branch of the Croton River two miles below the Bog Brook and Sodom Reservoir.*

A little later a similar but larger and more elaborate plant was built for treating the sewage of Danbury, Conn.

Each of these plants, as well as an installation for disinfecting garbage and ashes used for filling at and around Riker's Island, New York City, and also a plant used a short time for deodorizing purposes in the vicinity of one of the numerous outfall sewers in New York City,† produced the hypochlorite of sodium by an electrolytic process. At Brewsters and at Danbury common salt in solution was used, while wherever seawater was available the hypochlorite was made from that. This so-called Woolf process was used to treat sewage, or else the water about a sewer outlet, at Havana during the Spanish-American War.

The sewage-treatment plant at Brewsters has been continued in operation from that time to this, but with some enlargement.‡ The installation for treating water was used for a brief period only.

The Danbury sewage-treatment plant figured largely in a lawsuit brought by mill owners who claimed that the discharge of the sewage of Danbury was responsible for filling their mill-ponds with sludge and creating serious nuisances. The city attempted to show that the hypochlorite plant which it had established was all that was necessary to render the sewage unobjectionable, but the court concluded otherwise and under its orders Danbury installed intermittent sand filters and shut down the plant, which had been built at a cost of many thousand dollars.

A large amount of promoting work was done to exploit the so-called Woolf process, but aside from the installations we have named, little ever came of the process except that considerable quantities of "electrozone" a trade name given to the electrolytic produce in solution, were, we understand, bottled and sold as a rival to other ready-to-use commercial disinfectants.

Although put forward as the invention of Mr. A. E. Woolf, of New York City, in 1893, the process exploited by Mr. Woolf was similar to if not identical with the Hermite process, which was tried at Rouen, France, as early as 1889. Little more seemed to come of the Hermite than of the Woolf process, although reports came to this country from time to time regarding its use for treating the sewage of several French cities. Each inventor, substantially, was merely making use of the well-established value of "bleaching powder" or chloride of lime as a disinfecting agent, although much more was claimed for the respective "processes" by their exploiters. Each inventor, also, was merely using electric current to produce a well-known chemical agent.

In 1897 bleaching powder was used for disinfecting the water mains of Maidstone, England, after a typhoid epidemic, and in 1904-5 hypochlorite of soda was used at Lincoln, England, also in connection with a typhoid outbreak, to treat water going to filters, and also to sterilize the water-works reservoirs.

A few years ago, studies made by Phelps at Red Bank, N. J., and Baltimore, Md., and by Pratt and Kimberly at Camp Perry, O., and elsewhere, indicated that the hypochlorites of calcium and sodium might be successfully used to disinfect sewage effluents where a high degree of purity was required. These findings having been made widely known, they, in connection with various earlier experiments and attempted practical use of hypochlorites in water and sewage treatment, led Dr. J. L. Leal, of Paterson, N. J., and others associated with him in a lawsuit involving questions as to the fulfillment of contract requirements regarding the purity of the water-supply of Jersey City, to install a hypochlorite plant at the Boonton reservoir of the Jersey City Water-Works. That plant had a capacity of treating 40,000,000 gals. of water per day. Messrs. Hering and Fuller were associated with the installation, and at the meeting of the American Water Works Association, early in 1909, two of the members of that firm, in conjunction with Dr. Leal, presented a somewhat elaborate paper describing the Jersey City installation, and also giving an outline of the history and a statement of the underlying principles of the use of hypochlorites as water and sewage disinfectants. The large size of this plant, its evident great success in meeting the objects for which it was installed and the publicity given to the plant and the results obtained, together with what had recently been published regarding the use of hypochlorites as sewage disinfectants, all combined to give an impetus to the use of hypochlorites which has resulted in the installation of at least six or eight plants for the treatment of water within the last two years.*

Elsewhere in this issue we describe some experiments with hypochlorites recently made in a study of the possible emergency use of the Connecticut River for the water-supply of Hartford, Conn. We also publish articles in this issue describing the use of hypochlorites at Harrisburg, Pa., Nashville, Tenn., Quincy, Ill., Minneapolis, Minn., and Montreal, Que. Readers are referred to the several articles for details, but it may be noted here that at Harrisburg the hypochlorite is used with sulphate of alumina, sedimentation and mechanical filtration; at Quincy, with sulphate of iron and lime, sedimentation and mechanical filtration; at Nashville, with sedimentation alone, while at both Minneapolis and Montreal the hypochlorite is the only treatment given the water except such benefit as it gets from limited sedimentation in storage or distributing reservoirs. At both Montreal and Minneapolis the hypochlorite was introduced early in the present year to check typhoid outbreaks. There are two plants at Montreal: one at the city water-works and one at the works of the company which supplies outlying parts and suburbs of the city. Quite recent press reports state that hypochlorite is also being used at Toronto, on account of a sporadic outburst of typhoid there.

A reading of the several articles which we are publishing this week, combined with the outline sketch above given, will show conclusively, we think, that the hypochlorites of calcium and sodium are factors to be reckoned with in the future

*See Eng. News, July 13, 1893, for description of the plant for treating sewage, and July 27 (p. 65), Aug. 3 (p. 85), and Aug. 10 (p. 105), 1893, for brief notes on the installation for treating water.

†This plant was installed at the foot of Canal St., North River, to deodorize the water in the slip used by the Albany Day Boat Line. See brief note, Eng. News, Aug. 31, 1903 (p. 165).

‡It should be noted that the sewage at Brewsters, after hypochlorite treatment, was discharged into trenches in a meadow a few hundred feet from the river.

*See Eng. News, June 24, 1909, for abstracts of papers by Messrs. Leal, Fuller and Johnson on the Jersey City plant at Boonton, N. J.; and Eng. News, Sept. 16, 1909 (p. 308), for account of Mass. State Board of Health experiments with "bleaching powder, written by Messrs. Clark and Gage, and also for a brief reference to use of hypochlorite in connection with sedimentation, coagulation with sulphate of alumina and slow-sand filtration at Poughkeepsie, N. Y.; also see Eng. News, March 18, 1909, for experimental results on a practical scale with the use of hypochlorite by the Baltimore County Water & Electric Light Co., which supplies a number of suburbs of Baltimore, Md.

in the treatment of water. Their limitations should be distinctly understood. Fortunately, these remarkably effective disinfecting agents have been extensively studied by trained scientists during the past few years and for the most part are now being installed and used under the direction of competent men. Accordingly, there is little danger that such extravagant claims will be made for these agents as were put forward fifteen years ago.

The function of the hypochlorites is to serve as germicides. They are to be considered as disinfecting rather than sterilizing agents, although it is believed that for practical purposes hypochlorites, like well-operated filters under favorable conditions, will eliminate substantially all harmful germs from water. But hypochlorites have little effect in any other way than as germicides. For the removal of suspended matters, whether organic or inorganic, and for removing or reducing color, reliance must be placed on sedimentation, coagulation and filtration, or combinations of these. Moreover, except in emergencies, and under various special and local conditions, the use of hypochlorites will probably take a position secondary to other methods of treatment. They appear to be particularly valuable to supplement sulphate of alumina, whether used as a coagulant in connection with sedimentation, or whether used in conjunction with mechanical filters. When used with sulphate of alumina the hypochlorites reduce the requisite quantity of that agent more than enough to offset the cost of the hypochlorites, and thus materially decrease the total cost of chemicals. Hypochlorites also make possible increased rates of operation of mechanical filters, and lessen the frequency of washing and the quantity of wash water required in mechanical filtration.

One of the greatest points in favor of the use of hypochlorites is the simplicity and low cost of the dosing apparatus required for introducing these agents; another is the speed with which the necessary dosing apparatus can be installed.

The recent rapid adoption of the use of hypochlorites in the treatment of water leads one to wonder whether their use in treating sewage, modified from the earlier practice to suit later and more scientific information, may not become quite as popular and effective at an early date. It does not seem *probable* that this will be the case, but it is quite within the limits of possibility. For the present we shall look to see hypochlorites used ten to a hundred times for water treatment to once for sewage treatment. But with the increasing demands for the prevention of stream pollution, it may be well that the use of hypochlorites will be taken up soon and rapidly extended.

It is of interest in this connection to quote from a paper on "The Electrical Purification of Water," read in 1894, before the New England Water Works Association, by the late Prof. Thomas M. Drown.*

I would remark first, that there is nothing new in the application of electricity to produce the hypochlorites, except so far as the introduction of the modern dynamo enables us to generate electricity at a much less cost than formerly, and consequently to use it economically in industrial chemical operations. Second, there is nothing new in the use of sodium hypochlorite as a disinfectant, deodorizer and germicide. In the very valuable report on disinfectants published by the American Public Health Association in 1888, the hypochlorites were shown, experimentally, to be among the most effective of all disinfectants in their action; 1% of a solution of sodium hypochlorite, containing 6% of available chlorine, destroyed the "bacillus anthracis" in two hours; 2% of this solution was effective in 30 minutes.

Of the corresponding lime salt, or bleaching powder, the committee reported that a solution of this substance, containing 25% of available chlorine, was effective in a 1% solution in one or two minutes, and that a solution of one-fourth of this strength would kill "bacillus anthracis" and "bacillus subtilis" in two hours. The two hypochlorites of sodium and calcium were thus shown to be equally efficacious, the activity depending on the amount of hypochlorous acid present, or, in other words, of available chlorine. . . .

Finally, is it desirable in any case to treat a city's water-supply with a powerful disinfectant like the hypochlorites? When the question is put in this bold form I cannot think it will receive the approval of engineers and sanitarians. Leaving out of consideration the difficulties and cost of disinfecting effectively and uniformly the water-supply of a large city, the idea itself of chemical disinfection is repellent. Every city should jealously guard its water-supply. It is a sin against decency and good morals to permit the pollution of drinking water by sewage and house drainage. It is a still greater wrong, not to say a foolish blunder, to permit the open pollution of its drinking-water streams, and then systematically to disinfect them.

In cases where a water-supply has got into such a hopelessly bad condition that nothing will render it safe but disinfection by chloride of soda or chloride of lime, it is high time, I think, to abandon the supply, and in this opinion I feel sure most water-works engineers will coincide.

Such sentiments as those expressed in the last two paragraphs of our quotation, reflecting as they did both the scientific and popular opinion of the day, go far towards explaining why the hypochlorites have been so long in coming into practical use. And though both the scientific and popular attitudes have been materially changed in the past fifteen years, there are still many who are averse to placing reliance upon "a powerful disinfectant" for the purity of a public water-supply. But this need not prevent the use of hypochlorites for emergency use or as supplemental means of increasing the efficiency and reducing the cost of other and primary methods of water purification.

When Prof. Drown wrote the words we have quoted, there was in some quarters almost if not quite as strong a feeling against the use of sulphate of alumina as an aid to the mechanical filtration of water as he expressed against the use of hypochlorites as water disinfectants. Since then, careful and extended scientific experiments, coupled with equally scientific observations of the workings of scores of mechanical-filtration plants, have demonstrated to the satisfaction of practically all engineers and sanitarians that sulphate of alumina is a valuable and safe agent for use in water treatment. Similar experimental and practical evidence is now rapidly piling up in support of the opinion that the hypochlorites, too, may be used without danger in the treatment of water. Moreover, it should be noted that the quantity of hypochlorites used is minute compared with what was used in the earlier days, due largely to the fact that its use is now restricted to germicidal purposes, whereas formerly it was claimed by some that hypochlorites afforded a complete means of sewage purification.

Mechanical filtration did not come into its own until it was raised above the level of the commercialism and the patent litigation of the first 10 or 12 years of its history. The use of hypochlorites, though never subject to the same commercial and patent rivalry, also suffered at the hands of its early commercial exploiters and never gained a permanent footing in either water or sewage treatment until it was taken up by engineers and other scientists, its real possibilities determined without regard to commercial motives, and its use governed by its natural limitations, adapted to local conditions. There is a lesson in this to which many a man might well give heed.

*Reprinted in Eng. News, March 22, 1894. We may note here that articles on various early electrical processes for treating water, none of which passed beyond the trial stage, so far as we know, appeared in Eng. News, as follows: the Webster process, April 13 and Oct. 26, 1889; Hardie process, Feb. 2, 1889; process of Electrical Water Purification Co., May 17, 1890. In general, it was claimed for all these processes, as also for the Woolf electrozone process, that they would remove or assist in removing organic impurities, as well as destroy bacteria.

LETTERS TO THE EDITOR.

The Inventor of the Direct-Acting Explosion Pump.

Sir: My attention has been called to a letter under the above heading contained in your issue of Feb. 10, which appears likely to be misunderstood by your readers. Neither the writer of the letter in question nor myself can claim to be the inventor of the direct-acting explosion pump seeing that it dates back to 1868.

It is interesting to me to learn that another inventor has appeared in the field of operations to which I have devoted so much time and money; but it would seem that, although working in the same direction, we have approached the subject on very different lines. It is somewhat curious that I have regarded as unpractical and of little value any scheme for raising water by explosions in which the products of combustion become mixed with the water raised. I have also been careful to avoid the "violent" surging of the water piston which Mr. Smyth seems to regard as a "most highly desirable condition" and which results from allowing the explosion to act upon a relatively small water piston. I purposely choose a large mass of water so that the velocities are not too high and the water losses are consequently low.

Until after the publication of the letter referred to above I had not heard of Mr. Smyth, in any way whatever. No patents of his have been cited against my own in any country, and I may mention that I now hold patents in all important European and foreign countries and have a number of pumps constructed, and working up to units of six million gallons per day, with much larger pumps in hand.

The Humphrey pump which you described and illustrated in your issue of Dec. 2, last, differs so entirely from that described in Mr. Smyth's specification that, had it also been illustrated, your readers would have found it difficult to see any practical points of resemblance.

Yours faithfully,

H. A. Humphrey,
Consulting Engineer.

38 Victoria St., Westminster, London, E. C., March 18, 1910.

Impact Tests on Bridges.

Sir: The impact formula which was reported by the subcommittee on Impact Tests at the meeting of the Maintenance of Way Association* is as follows:

$$\text{Percentage Impact, } I = \frac{100}{1 + \frac{l^2}{20{,}000}}$$

where l is the span in feet.

The brief statement which was read by me before the meeting included not only the summary, which has been published,† but a preliminary general statement as follows:

The subcommittee on impact presents herewith a report covering the work of the past three years. This report is not to be considered a final one upon the subject, but the experiments which have been conducted have been fairly comprehensive, except for long spans, and it is the intent of the committee to present at this time a report which shall include a full account of the tests, the detailed results, and such a discussion of the same as appears to be warranted. Results of other tests have been briefly noted, but are not discussed. The subject of secondary stresses has not been touched upon farther than to present a few typical results obtained as incidental to the main experiments.

The subcommittee is not prepared at this time to recommend an impact formula for general use in specifications. It has seemed proper, however, in presenting the results of the tests made, to express the law of maximum impact, as thereby developed, by a formula, and this has been done. It should be noted that the impact percentages herein discussed relate to the variations in the axial stresses in the members, and do not include the element of secondary stress. The few results on secondary stresses noted herein, as well as measurements of such stresses by other experimentors, indicate that they are very important and require careful consideration in the selection of working stresses used in design. The element of secondary stress may or may not be included in a single formula with that of impact. This is a question which must be considered in making recommendations of an impact formula.

The full report will include some fifty pages of text besides tables and diagrams. I suppose it will appear in bulletin form as early as it can conveniently be printed.

F. E. Turneaure.

Madison, Wis., April 1, 1910.

Bonding Plaster Coatings to Concrete Walls and Ceilings.

Sir: The writer has done his best to secure information on plastering concrete surfaces and finds many opinions. For a number of years he has examined specimens of plastered work and talked with practical plasterers. During the past four years his own practice has been pretty well fixed and so far the results seem good and no reports have come of plaster peeling off.

It makes a slight difference, of course, whether the concrete surface is smooth or rough. The difference is in favor of a fairly uniform surface. Where depressions exist the excess of mortar has a tendency to "slump" out. It is therefore best to paint up very rough surfaces with an extremely dry mortar, using considerable pressure.

Dirt, just plain dirt, is the cause of much of the trouble experienced in making joints in concrete and making plaster coats stick. No matter how smooth the surface there is considerable surface porosity and the dirt film, not the smooth surface, causes trouble. Sour beer, vinegar or any weak acetic acid applied with a brush will clean the surface. A weak solution of hydrochloric acid is

*Eng. News, March 24, 1910, p. 349.
†Eng. News, March 31, 1910, p. 385, where, however, the formula was omitted. Prof. Turneaure is chairman of the subcommittee on Impact Tests of the Association's committee on Iron and Steel Structures.

good. Wire brush work is expensive and makes dust, something to be avoided. The richer the face, the more dust, and a rich concrete surface is never free from dust. Wash the surface, not scrub it, with a weak acid and then with water. At the time the mortar is applied the wall should be damp but not wet. Bricks are wetted to prevent a robbing of moisture from the mortar in the joints. Concrete to be plastered is wetted for the same reason, i. e., that the mortar coat may retain the moisture required to properly set it.

Having tried rich mixtures and lean mixtures, my practice is to use for a first coat a 1:3 mixture. It is

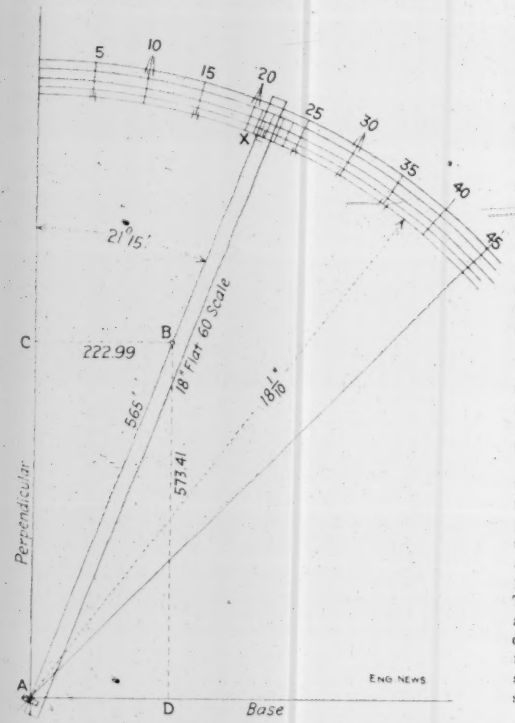

Fig. 1. A Home-made Trigonometer for Scaling Latitudes and Departures.

put on as dry as the mason will consent to handle it and applied with considerable pressure. The first coat is thin and scratched. The use of lime is forbidden. Ten per cent. of hydrated lime makes the mortar easy to work, increases adhesion slightly, weakens the mortar and more than doubles the time of setting. The addition of plaster of paris decidedly weakens the mortar and lessens the time of setting to a degree that embarrasses the plasterer. Too rapid setting lessens adhesion. A 1:3 fairly dry mixture applied with considerable pressure gives the best results.

The first coat should obtain its initial set and should be brushed with water before the second coat is applied. Like the concrete surface, it should be set and damp but not wet. No matter how many coats are applied, considerable pressure must be exerted and there must be as little working as possible. Every extra pass of trowel or float tends to consolidate the surface, draw the cement there and weaken adhesion to the next lower face. Painting with a thin cement grout on the surface of the concrete and even between mortar coats is of no benefit. The finished surface may be painted if desired. The best way to obtain the abominable shiny surface so many admire is to put on two cement grout paint coats and polish the last with a steel float. Such surfaces always craze.

Contractor.

Chicago, Ill., March 20, 1910.

Some Field Methods of Plotting and Recording Surveying Notes.

Sir: When you leave camp in the morning early enough so you can walk from two to three miles and get there just in time so there is daylight enough to see through the transit glass, work as long as you can see, walk back to camp, get off your snow shoes, have supper and a smoke, then as Chief of the party, appreciating that it is costing the company $65 or more per day to run your outfit, you start in on the day's notes to see "where you are at." You and the transitman with a couple of flickering tallow candles start to plot the line. Under these circumstances, I submit none of us is in the best condition to wrestle with logarithmic functions. It must be evident that any mechanical device that will save brain labor in such circumstances is a gain.

Trigonometers for this purpose can be bought in metal, but as a rule are worthless and can not be used with sufficient accuracy, so I was forced to devise one of my own. I do not claim anything new in the device except in its application. I provided myself with a board, made of clear white pine. This board was constructed of ½ × ½-in. pine strips glued and bolted together, making a board that will not warp, with a full size of 15 × 22 ins. On this I shrunk and glued a sheet of Whatman paper. I then graduated by successive compass intersections a "quadrant" to 15′ on an inside radius of 18.1 ins. (Fig. 1).

I then provided myself with a set of six 18-in. flat scales (paper scales will come close, but are not as accurate as the flat box wood) graduated from 10 to 60 parts to the inch, respectively, thus for any distance under 1,080 ft., you can scale to the nearest foot by using the largest scale admissible for the distance required.

At the zero of each scale I glued a short piece of isinglass (Fig. 1) and at *A* inserted a fine cambric needle. I then made a half dozen wood handle needle points of the usual kind.

In using the scale the first step in taking the course is to stick a needle point at the proper angle point *X* then lay on the proper scale against the needle point at *A* and *X* and set another needle point at the proper distance *B*. Then scale *B—C* and *B—D*, which are respectively the latitude and departure for the course and distance. It is evident that by inspection on this graduation to 15′ that the needle can be set within 2′ or 3′ of the correct reading.

In arranging transit notes for platting, under no circumstances will I permit a transitman to reverse the transit glass, except in running tangents. My order is "Keep the focusing screw either to the right or on top of the glass as the case may be," and for back sight read the angle around the plate (even though it be 359° 59′), then set down the angle, unclamp the spindle and repeat, setting down the result. If the second angle read is twice the first, go ahead, if not, find out why. Then taking the transit notes and, assuming the first course, by the angles and successive additions or subtractions all are reduced to bearings and tabulated. The difference between the sums of the right and left deflections must equal the difference between the first and last course making an easy and absolute check on the bearings.

In Fig. 2 I give the arrangement of the transit notes. I have added three extra columns marked "calculated," which of course would not appear ordinarily. In these latitudes and departures the total northing, accurately calculated to two decimal places, is 3.504.45 ft.; by the board it is 3.505.7 or out 0.75 of a ft.

Summed up by the board, Sta. 38 stands 0.75 too far north and 1.48 too far west; this is certainly very much closer than the ordinary field work is.

As for plotting, from the columns headed "Total" in the book, I locate at one laying-on of the scale the total distance north each point is of Sta. 0. Then drawing right angle lines to the north and south line at all points, I scale on these the distance each station stands east or west of this perpendicular. It will at once be seen that all cumulative errors are absolutely eliminated and each point located to a "needle prick," independent of every other, an absolute check being had on the plotting by scaling between the points, which must correspond to the distance between stations.

To show how beautifully this works out and reduces tedious calculations to a minimum, a lumber company desired to build a logging railway and when I suggested an engineer, they went up in the air (had had experience perhaps), "Great Heavens, if we wait for an engineer we will be through logging before he finds out where to build it." Passing over details, I said to the woods foreman, taking him to Sta. 0, "Set up stakes that you can see from one to the other, just where you would like to have the road." He put these at the points approximately 6 + 10 to 38. (Fig. 3.)

It took half a day for four men to chain the line and take the angles, from which I deduced Fig. 3. I then walked over it with the woodsman and located *X* as a

Sta.	B.S.	F.S.	INT.S.	H.I.	Elev	
PEG	4.60			147.72	143.12	
44		L	4.8		142.9	
5			5.7			
+85			7.5			
P	1.96	7.53		142.15	140.19	
6			5.8			
7			3.6			
8	3.77	8.46		137.46	133.69	
8	10.33		4.1			
9			6.2			
P.		2.97			134.49	CARRY-TO NEXT-PG
		18.76				
		10.33				
		8.43				
		8.12				
		134.49				

Fig. 4. Typical Left-hand Page of Level Notes.

production of the tangent back of Sta. 6 + 10 as Sta. 8 + 40. From the notes (Fig. 2) *X* is N. 21.15′ E. 840 ft. from O, and from the diagram (Fig. 1), it stands 782.9 N. and 304.5 E. of Sta. O. Now the point *Y* stands 3.504.7 N. and 474.9 W. of O, therefore *Y* stands 2721.8 N. and 779.4 W. of *X*. Therefore the bearing *X—Y* is N. 15° 59′ W., and the intersection angle at *X* is 37° 14′. I connected the tangent with a 5° curve, and incidentally missed *Y* a couple of feet. All the engineering, including marking the stakes for grading, cost about $75 and spoiled three days.

I accentuate the fact that by the table in Fig. 2, and by two simple subtractions, every angle point in the line is located, together with the distance east and north of any other point, thus giving the two sides of a right-angled triangle, which in every case are a north-south and east-west line.

I surprised myself in using this method. In addition to the logging track above described, I located about three miles more, using the same tactics. I put in stakes every 100 ft. on the preliminary, plotted them on a scale of 100 ft. to an in., walked over the line, taking side notes and projected the curves; and finally by scale located the stakes on the curves with a tape line by right angle offsets from the preliminary (all in heavy timber). After line was cleared I ran in the curves with a transit and found it not a couple of feet out of the way anywhere. I never ran a tangent to an intersection. This may be condemned as crude fudging and disgustingly unscientific. Admitted, but they hauled 10,000,000 ft. of logs over it to the mill and never had a wheel off but once and that on a defective frog. The cost of the engineering was a bagatelle. In the whole thing I never put a protractor on the paper or calculated an angle, nor cut down a tree to facilitate running the lines.

In calculating accurately the latitude and departure I minimize the figuring in a way I have never seen in the books. For example, take the first course given in the table in Fig. 2, I first get down

21° 15′	9.969420	(Cosine)
610	2.	(Index figure)
	9.589814	(Tangent)

Notice in setting down the figures I leave room for carrying in the logarithm of 610 and for the footing under the line over the tangent. Then turning to the logarithm table, I insert log. 610, foot up and complete the work thus

9.969420	
2.785330	
2.754750	Log of latitude.
9.589814	
2.344564	Log of departure.

This saves a couple of turnings to the tables, over the ordinary way of completing each separately.

Sta	to	S	Course	Dist.	Lat. & Dep. by Board N	E	W	Total Distance N-E or W from "0" N	E	W	Calculated Lat. & Dep. N	E	W
0.	6+10		N 21 - 15 E	610	568.5	221.		568.5	221.0		568.52	221.09	
6+10	9		N 5 - 7 E	290	288.7	26.3		857.2	247.3		288.84	25.86	
9	14		N 13 - 19 W	500	486.5	247.3	115.5	1343.7	131.8		486.56		115.43
14	16+50		N 27 - 51 W	250	221.		116.4	1564.7	15.4		221.05		116.79
16+50	20		N 61 - 18 W	350	168.		307.3	1732.7		291.9	168.08		307.01
20	38		N 10 - 3 W	1800	1772.		313.0	3504.7		604.9	1772.40		314.10
			O To 38 =	3800	3504.7		852.8 / 247.3 / 604.9						

FIG. 2. TYPICAL TRANSIT NOTES FOR TRAVERSE IN FIG. 3.

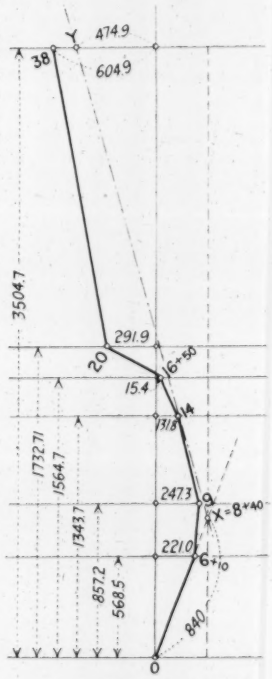

Fig. 3. Preliminary and Final Line for a Logging Road.

A word about keeping level notes. When I was on the Canadian Pacific Ry. some 20 years ago I had charge of the construction of 110 miles of the main line. The location level books were turned in to me without a mistake in the height of a turning point and never was an error carried beyond the page it occurred on. These levels were all run by one man, James Debrissey, whom I never saw but once in my life. I sat up and took notice and have never permitted level notes to be kept in any other way from that day to this. I have never seen this method in the books either. I give a page from actual work (Fig. 4).

It can be summed up in almost a word. The first sight on every page to be a *back sight* and the last one a *fore sight*. There is only one possible chance for error and that is in carrying the elevation over from one page to the next. You are always looking out for that.

Truly yours,

E. T. Abbott, M. Am. Soc. C. E.
General Manager Klamath Lake R. R. Co.

Pokegama, Ore., Jan. 7, 1910.

A Practical Coal Meter Made of a Chain.

Sir: The coal meter you illustrate in the issue of March 24, invented by Mr. O. D. Havard, is very interesting and reminds me of a somewhat crude arrangement made before 1904 for the same purpose.

This was made with a ¼ or 3/16-in. chain passing over a small wheel at the top of the 12-ft. vertical chute, 12 ins. in diameter, and the coal in descending carried the chain with it and so worked a register. I tried this experimentally and found it gave results within 2 or 3%, as I remember, but as the volumetric measurement of coal did not appeal to us the scheme was not put into practical use.

The above was installed or tried at the 33d and Market St. power station of the Philadelphia Rapid Transit Co., and the coal used was small anthracite. The original intention was to put small flights on the chain but it was found that the coal filling the pipe as it did, clung to the chain and moved it proportionately to its own movement without flights.

I am glad to see the idea has been carried farther as there are many places where such measurements will be of value.

Francis Head, M. Am. Soc. M. E.
Mech. Engr., Bureau of Water.

Philadelphia, Pa., March 24, 1910.

Traction Engines.

Sir: The abstract of my paper on "The Development of Traction Engines," in your issue of Jan. 27 contains several typographical errors, none of which are of importance. But in connection with the automobile manufacturers' formula it is stated that the formula is based on a piston speed of 250 ft. per min., whereas it should be 1,000 ft. per min. The denominator of the formula is 2.5 instead of 2.6.

Yours truly,

W. F. MacGregor,
J. I. Case Threshing Machine Co.

Racine, Wis., March 15, 1910.

Proposed State Regulation of the Practice of Engineering and Surveying in Ohio.

Sir: The General Assembly of the State of Ohio has before it a measure entitled "A bill to insure safe, economic and scientific construction, inspection and supervision of public work by establishing a State Board of Registration and Examination for Engineers and Surveyors, and regulating the practice thereof."

The Ohio Engineering Society has for a number of years been endeavoring to secure the passage of such an act. The original draft of the bill was prepared by a committee of the society, and indorsed by the membership in convention after prolonged discussion and slight amendment. The present form of the proposed law bears some resemblance to the original draft but has been subjected to so rigorous a treatment at the hands of the various committees of the legislative body as to be little more than recognizable.

The enthusiasm of the sponsors of the bill has been considerably dampened as they saw important provisions, one after another, cut out, but they are still pushing, feeling that the protection offered is better than none at all.

There is no doubt as to the necessity of such legislation in Ohio. The dignity and importance of the engineering profession is little recognized outside the large cities, and as a rule insufficiently even there, a condition largely due to the low standard of work set by many of its own members. The work of a few unscrupulous men who have utilized important public commissions to further their own dishonest ends has not improved matters.

The laws at present make the so-called "county surveyor" in reality designing and supervising engineer of all bridge and road work outside the cities, yet in a number of cases men have been elected to this office who were wholly ignorant of the first principles of engineering, and it is no uncommon occurrence for the average city of the state to entrust its engineering work to a man who at best is a fair transitman. Under the state laws the practice of medicine, law, dentistry, pharmacy, accountancy, stationary engineering and several other professions and trades are subject to complete regulation, but there seems to be a strongly grounded feeling that civil engineering and surveying should be open to any one who may choose to hang out a sign.

The bill now before the legislature, as amended, provides for the appointment by the Governor, subject to confirmation by the Senate, of a board of five civil or mining engineers, of at least five years' practical experience, and residents of Ohio for the same period, who shall have authority to issue certificates entitling the holders to practice civil or mining engineering or surveying. Not more than two members of the board may be teachers of engineering.

Any engineer or surveyor who has for three consecutive years, prior to the passage of the act, been engaged in active practice, is entitled to receive a certificate without examination. Graduation from a technical school of recognized reputation is to be regarded as equivalent to two years of active practice. The latter is thus defined:

> An applicant to have been in the active practice of civil or mining engineering, within the meaning of this act, shall have been engaged as a civil or mining engineer in designing or constructing work of such engineering character. . . . "The performance of the duties of a professor of civil or mining engineering in a technical school of high grade shall be considered equivalent to an equal number of years of active practice."

Certificates are also to be issued to those who pass a satisfactory examination.

Two classes of certificates are provided for. One for the "Registered Surveyor," who is thereby authorized to do surveying and platting and certify to the correctness of plats, descriptions, areas, etc., having to do with the laying out or conveyancing of land. The other for the "Registered Civil or Mining Engineer," who may thereunder do any class of public or private engineering work, including that of the surveyor.

The board may reject an applicant on grounds of dishonest application, fraudulent methods of practice, or incompetency. Any certificate may be revoked by unanimous vote of the Board, after the filing of charges, and proper hearing, for any violation of the provisions of the act, subject, however, to appeal to the courts.

The act does not become effective as regards regulation of practice until one year after passage, after which it forbids the holding of public office or employment as surveyor or engineer by other than the holder of a proper certificate, and after which no design or drawings "for any bridge, canal, waterway, highway, sewer, sewage disposal plant, filter, waterworks, ditch or mine, nor any estimate or specifications therefor," shall be filed for public record unless endorsed as approved by a registered engineer. Subordinates directly under the supervision of the holder of a certificate are specifically excepted from the provisions of the act.

The cost of the surveyor's certificate with or without examinations will be respectively $15 and $10, and of the engineer's certificate $25 and $15.

A fine of $50 to $200 is provided as penalty for unlawfully signing as registered engineer or surveyor, or for violation of any of the provisions of the act. Provision is made for organization and compensation of the Board, and the duties, powers and system of records are defined. The attorney general or any prosecuting attorney is required to conduct prosecution of any violation on request of the Secretary of the Board. All fines are paid into the state treasury to the credit of the fund created to meet the expenses of the board; this fund is to be provided wholly by the payments for certificates and such fines as may be collected.

It will be readily seen that this law will have little effect on conditions for some years. It is confidently believed by its advocates, however, that its effect will be to gradually improve the class of men engaged in engineering work in the state, and while neither the public nor the engineers will be protected as completely as might be considered wise by many, the bill is apparently as stringent a measure as the Assembly cares to pass; in fact, it is likely to be further emasculated, and may even fail of passage in any form.

E. F. Bradbury, M. Am. Soc. C. E.

Columbus, Ohio, March 30, 1910.

The Rates for Electricity of the Pasadena Municipal Electricity Works.

Sir: Commenting on an article in your issue of Feb. 10; believers in municipal ownership should be jealous of the correctness of returns and deductions to be derived from the operations of municipal undertakings. It looks as though Pasadena bookkeeping was "the common or garden variety" and very much open to criticism. For instance consider these figures:

	Yearly charges— As reported.	Should be.
Manufacturing	$12,742.78	$12,742.78
Distributing	4,046.83	4,046.83
Management	4,111.46	4,111.46
Depreciation & renewal fund (2½%)	.....	7,819.44
Bond interest	8,500.00	12,818.75
	$29,401.07	$41,539.26

Certainly my suggestion of 2½% is a very moderate renewal fund for boilers, engines, electric equipment, cedar poles, wire, arc lights, etc., and probably most engineers would suggest 3% to 4%. As regards interest on bonds, the amounts of bonds and rates of interest are given as "outstandings."

But these omissions do not form the serious indictment as compared with the charge for street lighting, which is very disproportionate to the commercial lighting. Who ever heard of $30,000 street lighting for a town of 40,000 population—no doubt it is a well lit town and from its character as a residential winter resort the commercial account would be rather small, but it is evident that street lighting has to carry the enterprise, and in order to make a good showing the citizens are charged up with a heavy street lighting account, at high rates.

I have, similarly, seen accounts in which the fire department rentals of hydrants were boosted so that the citizens got cheap water.

Would it not be in order to have a little more light on the Pasadena accounts and perhaps Mr. Koiner will explain further the cause of the $30,000 charge for street lighting and at very high rates per KW.-hour.

Yours truly,

C. B. Smith.

Smith, Kerry & Chace, Confederation Life Bldg., Toronto, Canada, Feb. 26, 1910.

[In the article from which Mr. Smith makes the foregoing deductions the annual interest charge for the year ending July 1, 1909, was stated to be $8,500. It should be said in fairness that this was all the interest then due and was made up as follows, according to the report quoted:

Interest on 1906 issue ($18,750 in 4%s)	$4,750
Interest on 1908 issue ($50,000 in 4½%s)	2,250
Interest on 1909 issue ($150,000 in 4½%s), two mos.	1,500
	$8,500

The report further shows that the $118,750 outstanding of the 1906 issue was reduced during the year to $115,625, and the 1908 issue to $48,750, making the total of bonds outstanding $314,375, as shown in the extracts in our article of Feb. 10.

The street lighting was furnished by 306 arcs, 553 — 40-c. p. and 25 — 10-c. p. tungsten incandescent lamps, 56 — 32-c. p., 124 — 16-c. p. and 1,488 — 8-c. p. carbon incandescent lamps. As the manager's report stated that the arcs were burned 2,662.41 hours in the year, it is fair to assume that all street lamps were burned about that long. Then the total output absorbed by the lamps above enumerated is approximately 593,200 KW.-hrs. (346,000 KW.-hrs for arc and 247,200 for incandescent lighting). For this $30,563.56 was paid, while the amount which would be charged private consumers for such service, according to published rates, appears to be approximately $24,000.

From authorized statements, published in local newspapers, in reply to criticisms made on the operation of the plant, it appears that the plant received for public lighting an average of 4.4 cts. per KW.-hr. for energy furnished during the first six months of the fiscal year 1909-10. During the same period it received an average of 5.42 cts. per KW.-hr. from commercial sales. As there is good reason to believe that these statements fairly represent the actual condition of affairs, the discrepancy, between what the city actually paid and what a private customer apparently would have had to pay must be laid to insufficient information in the earlier report to justify the comparison made.

Mr. C. W. Koiner, General Manager and Electrical Engineer of the Municipal Lighting Department of Pasadena has replied to Mr. Smith's comments in the following letter. From the two reports mentioned by Mr. Koiner we reprint a few summarized tables.—Ed.]

Sir: I have to state that Mr. Smith, not being familiar with all the circumstances in connection with the municipal lighting situation in Pasadena, was, of course, unable to draw the right conclusions. I am enclosing here-

with an eight months' statement, which has just been issued by this department.

The statement published in Engineering News covered a period of twelve months for street lighting and there were only eight months of this during which we did any commercial business. In fact, our lines were not completed over the entire city at that time, and we took what business we could procure along our lines. At the same time the opposition company, whom the city had to fight, offered flat rates and attempted to give away current to prevent the city securing a load. This company was given an opportunity to sell out to the city, but it would not consider the proposition; hence we are engaged in a strenuous fight for business. The city has at this time 2,010 customers.

In addition to the eight months' report published here recently you will find enclosed a report for our last month's operation, which may be of some interest.

Yours truly,
C. W. Kolner, General Manager.

Pasadena, Cal., Mar. 19, 1910.

Pasadena, Cal., March 15, 1910.

To the Honorable, the Mayor and City Council of the City of Pasadena:

Gentlemen: Submitted herewith please find report covering the operation of this department for the eight months ending Feb. 28, 1910, together with a statement of the construction (bond) account to March 1, 1910.

The system is now furnishing: 300 arc lamps, 879 40-c. p. tungstens, 25 60-c. p. tungstens, 11 80-c. p. tungstens, 57 32-c. p. carbon incandescents, 128 cluster posts (1, 16-c. p. and 12, 8-c. p. each—Ed.); all city buildings and 7,984 private consumers.

Receipts and charges		$46,773.49
Expenditures		23,901.51
		$22,871.98
Interest on outstanding bonds for eight months	$8,545.84	
Allowance for retiring bonds for eight months	5,416.64	
Balance applicable for construction or depreciation	8,909.50	
	$22,871.98	

Works operated (hours), 5,821.
Arcs lighted (hours), 1,764½.
Output KW. hrs. (net), 1,153,246.
KW.-hrs. per bbl. oil, 118.32.
Manufacturing cost per KW.-hr., 0.013.
Distribution cost per KW.-hr., 0.0077.
Total cost manufacturing and distribution per KW.-hr., 0.0207.
Operating expenses 51% of gross income.

EARNINGS.

Street lighting, 8 months	$21,541.74
City buildings	795.33
Charges for commercial light and power	24,436.42
	$46,773.49

EXPENDITURES.

Manufacturing:		
Fuel oil	$9,162.51	
Repairs to plant	1,268.43	
Wagons	3,307.61	
Miscellaneous	1,260.42	
Total		$14,998.97
Distributing:		
Operation of public lamps, etc.	$1,814.59	
Operation of commercial lamps, etc.	1,806.74	
Repairs to lines	786.39	
Total		$4,767.72
General:		
Salaries	$3,158.79	
Office expense and supplies	745.07	
Insurance	75.00	
Advertising and soliciting	395.97	
Uncollectable accounts	59.99	4,434.82
		$23,901.51

Pasadena, Cal., Mar. 11, 1910.

To the Honorable, the Mayor and City Council of the City of Pasadena.

Gentlemen: Attached hereto please find report of expense for material, supplies and labor for operating the Municipal Lighting Works Department for the month of February, 1910:

Receipts and charges		$7,666.54
Expenditures		3,488.39
		$4,178.15
Interest on all outstanding bonds	$1,068.23	
Monthly allowance for retiring bonds as they become due	677.08	
Balance applicable for depreciation or construction	2,432.84	
	$4,178.15	

Works operated (hours) 682.
Arcs lighted, 206¼.
Output, KW.-hrs. (net) 176,952.
KW.-hrs. per bbl. oil, 124.4.
Mfg. cost per KW.-hr., 0.01188.
Distribution cost per KW.-hr., 0.00788.
Total cost per KW.-hr., 0.01976.
Operating expenses, 45% of gross income.
Charges for construction during month, 9,410.73.

Earnings:

Street lighting	$2,902.93
City buildings	106.96
Charges for commercial light and power	4,656.65
	$7,666.54

Expenditures:

Manufacturing	$103.90
Distributing	805.61
General	578.88
	$3,488.39

A High Duty Record for a Pumping Engine at Atlanta, Ga.*

Sir: As the subject of pumping engine duty seems to be of current interest, the writer ventures to send you abstracts from a report of test conducted by him, of a 20,000,000-gallon, vertical, triple-expansion pumping engine, built for the Atlanta water-works by the Wisconsin Engine Co. This test was of 15 hours' duration, and was made Feb. 6, 1910.

The plunger displacement was at the rate of 21,212,000 U. S. gallons per 24 hours and the duty attained per 1,000 lbs. of dry saturated steam was 176,698,300 ft.-lbs.

The gross steam used by the engine and appurtenances, as determined by discharge from the receivers, jackets, etc., and from the air pump of the surface condenser, was 170,274.5 lbs. This quantity was corrected for moisture as indicated by throttling calorimeter, by deducting 1.93% entrainment, leaving the net steam used by the engine 166,961.8 lbs.

The contract required that the water be delivered into the main in the street in front of the building at a pressure of 120 lbs. The water being pumped was taken from the clear-water basin of a filter plant, which had an elevation above the engine room floor, and after all elevations had been taken and accounted for, it was found that the head upon the engine suction closely approximated the elevation of the water pressure gage upon the engine gage board. After all corrections were made, including elevation of this gage above the center of the main in the street, the working load was determined to be 115 lbs. upon the engine water pressure gage, for the test run.

The average of 46 readings of the pressure gage, gave 115.6848 lbs. as the working load at that elevation. This corrected for the elevation of the suction water column, gave a net load against the plungers 266.653 ft.

Weight of water at observed temperature of 44°.	62.42 lbs.
Weight of water delivered by engine per rev.	4.948 lbs.
Revolutions of engine during test	22,359
Total weight of water pumped	110,637,669 lbs.
Total ft.-lbs. (weight × head)	29,501,866,351 ft.-lbs.
Duty per 1,000 lbs. dry steam	176,695,300 ft.-lbs.
Work of pumps	933.056 water HP.
Work of steam cylinders	1,060.693 I. HP.
Mechanical efficiency	93.62%
Steam per indicated horse-power	10.49 lbs.
Duty per 1,000,000 B. T. U.	160,634,834 ft.-lbs.
World's record, B. T. U. basis	163,925,300 ft.-lbs.
Coal duty at 7½ lbs. evaporation	132,523,725 ft.-lbs.
Coal duty at 8 lbs. evaporation	141,358,640 ft.-lbs.
Coal duty at 9 lbs. evaporation	159,028,470 ft.-lbs.
Diameters of high pressure cylinder	33 ins.
Intermediate cylinder	62 ins.
Low pressure cylinder	94 ins.
Water plungers, average	31.1 ins.
Stroke of all, average	60.1 ins.
High pressure cylinder, indicated	344.795 HP.
Intermediate cylinder, indicated	371.635 HP.
Low pressure cylinder, indicated	344.263 HP.
Total indicated horsepower	1,060.693
Average power for the three plungers against the load	331.0186 HP.
Total work of main pumps	993.056 water HP.

Before the test was started, the various substances and instruments were carefully considered, and their respective values agreed upon, so that a unanimous decision could be reached by the parties to the contract, when the test was finished.

The following additional figures are of interest:

Average steam pressure during test, gage	152 lbs.
Contract speed of engine	24 r. p. m.
Speed during test run, average	24.8433 r. p. m.
Average water pressure in city main in street, during test run	121.7748 lbs.
Av. pressure in 1st receiver, above atmosphere	40.84 lbs.
Average pressure in 2d receiver, below atmosphere inches of mercury	2.5739 inches
Benefit of vacuum in low pressure cylinder, average for stroke	11.545 lbs.
Absolute initial pressure in high pressure cylinder at point of cut-off, average	160 lbs.
Absolute terminal pressure in low pressure cylinder, average	5.6 lbs.
Ratio of expansion	28.57 to 1
Absolute mean effective pressure by hyperbolic logarithm	24.374 lbs.
Theoretical M. E. P. deducting counter pressure	21.3557 lbs.
Work on low pressure cylinder basis, from logarithm and counter pressure	1,115 HP.
Efficiency of indicated horsepower compared with logarithm effect	95.17%
British thermal units per indicated HP. minute, 190.8816 B. T. U.	
Thermal efficiency of the pumping engine	22.22%
World's record for thermal efficiency	22.80%

This pumping engine is of the vertical, triple expansion, crank and fly-wheel type. An illustration of a very similar machine, appeared in Engineering News of Feb. 3, 1910, p. 120. There are three outside-packed plungers, one plunger directly beneath each steam cylinder, and with each plunger rigidly connected to one of the steam pistons. The three plungers are connected together in one pumping unit, by means of connecting rods, and cranks set at angles of 120°.

The main pumps are of the separate plunger barrel pattern, which is no doubt the best arrangement for this class of machinery. The main pump valve chambers are entirely separate and apart from the plunger barrels. The engine is self-contained, no foundation being required, aside from a solid concrete bed, laid upon the subsoil, and supporting a cast-iron sole plate, extending beneath the water end of the machine. The main pillow blocks are supported at the level of the engine room floor, upon the valve chambers.

The steam end of the pumping engine has its high pressure cylinder fitted with Corliss-type steam and exhaust valves. The intermediate and low pressure cylinders are fitted with Corliss-type steam valves, and with poppet exhaust valves. This arrangement of valves affords about the minimum of clearance in the steam cylinders.

The fly-wheels are two in number and weigh about 65,000 lbs. each. The main framing is of the single "A" box type, with bored guides for the cross head shoes.

The condenser is of the surface type, placed in the suction main leading to the main pumps. The air pump is driven from the low pressure plunger head, and has the full stroke of the engine.

Yours respectfully,
Chas. A. Hague.

52 Broadway, New York City, March 23, 1910.

*It is no more than fair to state that this engine had been operated against its load only about 1,000 hours, and intermittently at that, from the time it was first started to the time of official test. Longer opportunity to get down to smooth working bearings, would very likely have resulted in higher mechanical efficiency and a higher duty record.

A Test of High Duty Pumping Engines at the Philadelphia Water-Works Pumping Station, Lardner's Point, Frankford, Pa.

Sir: Referring to the high duty records recently attained on large pumping engines as described in your issues of Feb. 3 and March 10, it may be of interest to your readers to know of the results attained in a recent test of some large engines recently installed in the Lardner's Point water-works pumping station in Philadelphia. There are three pumping stations at this place.

Station No. 1 was formerly known as Frankford Station, and contains boilers and pumping engines that have been in service a great many years, and which are now retained only as a reserve, the machinery being entirely too expensive in operation to keep in regular service.

Stations No. 2 and No. 3 are new pumping stations throughout; the buildings and equipment therein were constructed for the new filtration distribution system. Each consists of an engine house and boiler house. The two stations are contiguous and for all practical purposes are one station. In engine houses No. 2 and No. 3 are twelve vertical triple-expansion self-contained pumping engines, each of 20,000,000 U. S. gallons daily capacity, designed for a normal head of 225 ft., but capable of operating economically against heads ranging from 180 to 280 ft.

These twelve pumping engines, all substantially alike, were designed by the writer and were constructed and installed by The Holly Mfg. Co., of Buffalo, N. Y.

The water which these engines pump is supplied from the filtration plant of the city of Philadelphia at Torresdale, between two and three miles from the Lardner's Point pumping station, through an underground tunnel leading from the Torresdale filtration plant to the Lardner's Point pumping station. It is pumped from the station through a number of delivery mains, ranging in diameter from 48 to 60 ins., to various parts of Philadelphia.

Ordinarily, eight or nine of these engines discharge the water through two 60-in. delivery mains, which are so connected together as to form substantially one main of twice the capacity of a 60-in. pipe, against a head of approximately 185 ft. The other three or four engines ordinarily pump through a separate pipe system against a head of approximately 275 ft.

At 8 o'clock in the morning of Dec. 10, 1909, at a point about 1,000 ft. from the pumping station, a 12-ft. length of the 60-in. delivery main, into which eight of these pumping engines were at that time delivering water at the rate of over 160,000,000 gallons per day, against a head of approximately 185 ft. split from end to end, instantaneously reducing the resistance against the pump plungers from full load to almost no load. Five of these pumping engines were in Station 2, and three in Station 3.

All of the pumping engines were under such perfect control of the governors and automatic safety devices that only one of the eight attained sufficient speed to make the automatic shutting-down device operative, the result being that one engine stopped automatically, and the other seven ran at a uniform speed until gradually closed down by the employees at the station. Absolutely no damage was done to any of the engines or to any of the machinery in the pumping stations. The broken 60-in. delivery main was promptly repaired and the engines were all in service again the afternoon of the same day the accident occurred.

This is undoubtedly the first time the governing apparatus of so many pumping engines has been subjected simultaneously to a test of this character. That none failed to work properly and satisfactorily speaks louder than words for the reliability of the governing apparatus.

One of the twelve engines was officially tested on March 9-10, 1910, and developed a new high duty record for pumping engines fitted with attached jet condenser, on a 24-hour trial.

The test was made under the general direction of Mr. Fred C. Dunlap, Chief of the Bureau of Water, Philadelphia, Pa., and was conducted by Francis Head, Mechanical Engineer of the Bureau of Water, representing the city of Philadelphia, and by Edgar G. Hill, Mechanical Engineer, representing The Holly Mfg. Co., each assisted by a corps of capable and experienced observers.

During the test, readings of the water pressure were taken every five minutes from a correct pressure gage checked by a mercury column. A complete round of observations was taken every 15 minutes. All readings and weights were checked by two observers, one representing the city, and one the company. An eight-hour boiler leakage test was made before the duty trial, and also immediately after the duty trial.

A summary of the principal results is given below:

Number of engine		604
Size of engine..30 ins., 60 ins., 90 ins. × 33 ins.	×	66 ins.
Duration of test, hours		24
Steam pressure at throttle, lbs.		180.25
Total water pressure, lbs.		95.74
Vacuum, inches mercury		27.68
Total revolutions during test		28,989
Average revolutions per hr.		1,207.875
Average total feed water per hr., lbs.		9,265.625
Feed water corrections, average per hr., lbs.:		
Drips from steam header and steam separator	135.27	
Boiler leakage, average of two tests	196.39	
Total		331.66
Average total steam per hr., delivered to engine, lbs.		8,933.97
Entrainment, per cent		1.135
Average total dry steam per hr. delivered to engine		8,832.56
Water pumped per 24 hrs., gallons		21,218,788
Duty per 1,000 lbs. steam delivered to engine.		182,382,200
Duty per 1,000 lbs. dry steam		184,476,200

Very truly yours,

D. A. Decrow,

Chief Engineer Holly Mfg. Co.

Buffalo, N. Y., March 24, 1910.

Notes and Queries.

In an editorial note in our issue of March 24, 1910, on the present agitation over locating a new courthouse in the City Hall Park, New York City, we stated in effect that the land on which the general post-office is located had been occupied by the General Government rent free. It appears that this was an error, and that the portion of the City Hall Park in which the post-office is located was bought by the United States in 1867 for $500,000. The corresponding deed conveyed 65,259 sq. ft. of land, or something over an acre and a half.

The following is an extract from a letter sent to the Committee on Public Education of the New York Assembly, opposing the Raldiris Bill to establish a system of official licenses for the practice of Civil Engineering and restrict these licenses to graduates from engineering colleges:

If Mr. Raldiris were a member of the American Society of Civil Engineers, I believe he would not consider his bill of any value. If the law required that no engineering structure should be built without the cooperation of a licensed engineer, we might have some grounds for a bill for licensing engineers. But to-day nothing prevents a man from building himself any engineering work, unless it is a building or similar structure in a city.

I have practiced in many states in this country as well as abroad, and I have yet to find, in meeting with engineers, any who considered it necessary for the state to license engineers. If, however, the Empire State should set an example, and other states should follow, some of the older engineers might be placed in the absurd position in which a distinguished New York physician found himself. He was enjoined from practicing during the summer in New Jersey by an upstart with a New Jersey physician's license, but with little experience and no reputation.

Concrete Piles.*

Concrete piles may be divided into two general classes, "cast and driven," and "cast in place." The "cast-and-driven" type are always of reinforced concrete, are molded prior to being placed in the ground, and are driven like wooden piles. "Cast-in-place" piles are usually, though not necessarily, without reinforcement, and the pile hole is made artificially and afterward filled with concrete.

The first concrete piles were made in France, by Mr. Hennebique, in 1896, and were of the cast-and-driven type. During the following decade, this general type was developed by different engineers, and is now more generally used in Europe than any other. In the United States, the Gilbreth (patented 1908), Chenoweth (patented 1904) and Raymond (patented 1896) are the best known varieties of cast-and-driven piles.

In 1890, A. A. Raymond, of Chicago, Ill., conceived the idea of a cast-in-place concrete pile, a further development of the caisson principle. His first patents were issued in 1896, and the first piles driven in 1901. The Simplex pile, patented in the United States in 1903, is of the cast-in-place type, though differing from the Raymond in method of construction. These two systems have thus far been used, in this country, to a much greater extent than cast-and-driven piles, though owing to our comparatively recent development in concrete construction, our total use of concrete piles is less than the European. Prior to 1900 practically no concrete piles had been used in the United States.

CAST-AND-DRIVEN TYPES.—The chief objection to cast-and-driven concrete piles has been based upon the possibility of injury to the reinforced concrete during driving. Most of the systems care for this by jetting the piles, or by the use of a special driving cap designed to protect the head of the piles. The patents covering these various systems refer principally to the driving caps, the arrangement of the reinforcement, provision for jetting, method of molding, and the shape of the pile itself. All of the items entering into the manufacture and placing of cast and driven piles may be varied to suit the local conditions.

The Gilbreth pile is a corrugated, tapering pile, reinforced with Clinton electrically welded fabric, of ⅜-in. wires 3 ins. on centers longitudinally, and ½-in. wires 12 ins. on centers transversely, or other reinforcement of suitable design. The piles are made of any shape desired, but are usually octagonal, tapering from 16 ins. diameter at the butt to 11 ins. at the tip, with corrugations running the entire length. In case the piles are intended to project above the ground, the corrugations may be stopped at the ground level. The taper is designed to develop the bearing power of the soil throughout the entire length and the corrugations are added to form outlets for the water from the jet. A 2-in. hole is made through the center of the pile for jetting.

The piles are cast horizontally in wooden forms and allowed to set for various periods; in some instances they have been driven after seven days. They are handled by an ordinary pile driver and jetted into place; the weight of the hammer resting on the driving cap helps to force the pile down. The special driving cap consists of a steel shell with wooden blocks at each end between which are pieces of hose and rope to form a cushion. When the pile has been jetted nearly into place the hammer and cushion cap may be hoisted, the jet removed, the cushion cap replaced, and the pile driven to the desired penetration per blow.

The Chenoweth pile is round and is made without forms by a special machine. It is reinforced with a coiled sheet of wire netting and longitudinal steel rods placed near the surface, at equal distances apart. The driving cap is a steel cylinder, with a diaphragm in the center. The lower end of the cylinder fits on the head of the pile, and the upper end receives the wooden driving block. A cushion of rope is used between the head of the pile and the diaphragm, while a cushion of sawdust is placed between the diaphragm and the driving block.

The piles are driven with an ordinary pile driver and an extra heavy hammer. The concrete used is generally a 1:2:3 mixture of Portland cement, sand and screenings or broken stone.

CAST-IN-PLACE PILES.—In the Raymond system, a heavy mandrel, called a core, is surrounded by a casing of sheet steel and the whole is driven into the ground by means of a steam hammer. When the desired depth is reached the core is collapsed and withdrawn from the hole, leaving the steel shell in the ground as a form for the pile. The concrete is then shoveled into the hole and the pile is complete. The sheet steel shell is made heavy enough to withstand the pressure of the earth along the sides of the hole, and protects the concrete from mixture with the surrounding earth. These shells are usually made in 8-ft. sections, and each section laps the other 6 ins. The point section is first placed on the core, then the next above, etc., until the core is covered. This brings the lap on the outside and each section is pushed into place as the driving proceeds.

After the core is withdrawn and before placing the concrete, the interior of the shell can be inspected, by means of an electric light, by reflected light from a mirror, or by the light reflected from the surface of water thrown into the hole. The 20-ft., 25-ft. and 30-ft. cores taper from a 6-in. point to a 20-in. head; the 35-ft. from an 8-in. point to a 20-in. head; and the 40-ft. from a 10-in. point to an 18-in. head. This is a greater taper than is found in any other type of concrete pile, and is as great as practical limitations will permit. The diameter of the head, and the taper, are designed to provide a large bearing surface for the concrete footings, a large surface area for frictional bearing and a maximum compression of the earth surrounding the pile. While the Raymond pile is rarely reinforced, reinforcement may be added by lowering a fabricated skeleton into the shell before the concrete is placed. Rods may also be placed by churning them through the concrete after the shell has been filled.

The Simplex system consists in driving an extra heavy iron pipe into the ground with a special point to exclude the dirt. When this pipe is driven to the proper bearing, a drop-bottom bucket, filled with concrete, is lowered to the bottom and dumped. The bucket is then removed, a heavy weight lowered into the pipe, and the pipe raised nearly to the top of the concrete, the weight being repeatedly dropped on the concrete, thus forcing it out of the end of the pipe. Another bucket of concrete is then placed in the pipe and the operation repeated until the pile is formed.

Two kinds of points are used, depending upon the character of soil through which pipe is driven. In soft soil, a cast-iron point closes the end of the pipe. In stiff clay, jaws are attached by hinges to the bottom of the pipe, and automatically open to permit the concrete to flow through them as the pipe is raised. The pipes are extra heavy steel, banded where necessary, and are made up in sections of varying lengths to suit the length of pile required. They are cylindrical and are usually 16 ins. in diameter. The only limit to the length of pile, which can be driven by this method, is the ability to drive and pull the pipe. In stiff, non-water bearing, or clay soils, where the ground has no tendency to flow, this is the cheapest system of installing concrete piles.

ESTIMATED COMMERCIAL USE.—After careful inquiry it is estimated that about 2,500,000 lin. ft. of cast-in-place piles are now in use in this country. No definite data in regard to the use of cast-and-driven piles could be obtained, but the quantity is believed to be less than 250,000 lin. ft.

ADVANTAGES OF CONCRETE PILES.—Wooden piles are loaded to about 15 tons, and while many engineers think they are capable of carrying greater loads, this load represents conservative practice. The loads allowed on concrete piles vary from 25 to 50 tons each, depending upon the nature of the ground, and the system used; in European practice they are loaded to 40 or 50 tons each, and there are records of piles loaded to 80 tons. The best American practice, however, limits the load to 35 tons.

To prevent decay, wooden piles are cut off at low water and the foundations must be carried down to this level. The tops of concrete piles may be at any elevation desired and the depth of the footing course will be governed by the spacing of the piles, and the weight of the superstructure. In this way the expensive excavation for deep footings, as well as the cost of the additional depth of footings, is saved.

Concrete piles cannot be driven as rapidly as wooden piles, but since the load per pile is greater, their use may result in a saving of both time and cost. If concrete piles are used, no delay is occasioned by waiting for the arrival of the piles; wooden piles are difficult to obtain in certain parts of the country, but sand, stone and cement are generally available. There is no reason why concrete should vary in quality, while it is becoming more and more difficult to obtain wooden piles of a standard grade.

A wooden pile rots if alternately wet and dry and is subject to attack by the teredo and other wood borers, when placed in salt water. The elevation of ground water fixes the point of cut-off, but the building of deeper sewers in the neighborhood may lower the ground-water level, and when this occurs wooden piles are subject to decay.

POSSIBLE DISADVANTAGES OF CONCRETE PILES.—The argument most frequently urged against concrete piles is their cost. The cost per lin. ft. is certainly greater for a concrete pile than for one of wood, but when the saving in excavation, size of footings, number of piles and time of completion, are taken into consideration, this additional cost per foot is discounted. Generally when these items of cost are considered, they should show a decided advantage in favor of concrete piles.

The length of cast-in-place piles is limited, but piles cast and driven may be of any length. When driven with a core or pipe, the maximum length thus far attained is 45 ft.; no cores have been made longer than this, while the difficulty experienced in pulling the pipe, renders a greater length impracticable.

Cast-and-driven piles require careful handling if made in long lengths, as cracks cannot be readily detected, and the piles should preferably be allowed to set several weeks before using. If made on the job this delays the work, while if made in distant yards, the exact lengths are hard to gage, and the piles may be either too short or too long. As the operations of lengthening or shortening cause delay and expense, it is very desirable that the penetration be determined before the piles are brought to the site.

If shells are used, they should be made of steel heavy enough to withstand the side pressures of soft ground or the shearing effect of boulders or gravel. Some objection has been made to dropping concrete through the longer lengths of a cast-in-place pile. As a wet mixture is used, and the concrete is deposited by shovels, it should be properly graded when the pile form is full.

The driving of an adjacent pile tends to settle the concrete in the pile just filled, thus forcing out the air

*From the report of the Committee on Masonry, presented at the annual meeting of the American Railway Engineering and Maintenance of Way Association, Chicago, March 15-17.

bubbles, and while distortion may occur, particularly in the case of piles driven without a shell, this danger has not been manifest in actual practice.

In cast-in-place piles, unprotected by a shell, it is impossible to inspect the integrity of the pile. After the concrete leaves the end of the pile, it is subjected to the action of the surrounding ground without protection. Soils bearing running water may carry off a part of the cement, leaving the coarser aggregates; very dry, porous soils may absorb the water needed in setting; in either case the bearing power of the pile would be affected.

As in the case of peeled wooden piles, a smooth exterior offers less resistance to movement than a rough one. However, when the attempt is made to increase the skin friction by cementing the pile to the earth in which it is cast, a question arises as to how much the pile itself is weakened by an admixture of earth. The necessity of reinforcing concrete piles in certain soils is apparent, and the greater the dependence placed upon point bearing, the greater the necessity for reinforcing.

The bearing power of all piles is dependent upon a combination of two elements: point bearing and skin friction. Point bearing depends upon the class of material upon which the pile rests, the area of the point, and the material of which the pile is made. Skin friction depends upon the compression of the earth through which the pile is driven, and all the earth, from the top to the bottom of the pile, adds to its bearing power if subject to compression.

Probably the bearing power of every pile driven is dependent upon both the above factors, the percentage of each varying with the character of the soil and the shape of the pile. The larger the point, and the less the taper of the pile, the more point bearing is depended upon, and vice versa. A pile depending upon point bearing, acts as a column, supported more or less at the sides, and any pile acting as a column should be reinforced. When subjected to bending stresses or placed in semi-liquid or shifting soil, piles should be reinforced. Piles, whose method of manufacture precludes the insertion of reinforcement, are not adapted to use in service which requires action as a column.

An experiment was made to ascertain the distance to which a pile compacts and otherwise influences the surrounding soil and also how the earth acts under the lower end of a pile. A box about 1 ft. deep and 3 ft. square was constructed with a removable side. Sand of ordinary consistency was dumped into the box and compacted to a moderate extent. The side of the box was then removed and the surface divided into squares by shallow grooves filled with white sand. The side was replaced and bolted in position and a block of wood driven into the sand in exactly the manner in which a blunt pile would be treated. The effect of the penetration of the pile was shown by the distortion of the white lines, and it was interesting to note the distance below the block to which the earth was influenced and how the sand was actually bulged upward to some extent.

Marine Steam Turbines With the Melville and Macalpine Speed-Reducing Gear.

The Melville and Macalpine speed-reducing gear, to be interposed between a marine steam turbine and the propeller shaft to allow both turbine and propeller to be designed for operation at maximum economy of the type employed, has already been described in this journal.*

Readers will recall that the device comprises essentially a pair of helical toothed pinions, meshing with a corresponding pair of gears, and a "floating frame" which carries the bearings of the pinion shaft and is supported at a single point midway between these bearings. This support, as as been shown in detail, is of the nature of a web or stiff diaphragm which allows the floating frame and the pinions to oscillate, somewhat restrictedly, in a vertical plane through the axis of the pinions. This arrangement, it will be remembered, insures uniform contact of the teeth and makes the gears self-adjusting for the equalizing of abnormal strains due to small inaccuracies of machine work, of adjustment, etc. It will also be recalled that the efficiency, as shown by tests on a 6,000-HP. gear actually constructed, reached the surprising figure of 98.5%, due probably to the accurate machine work, to the effectiveness of the "floating frame" in promoting uniform tooth bearing and to the very high speed of operation.

We have remarked before that, if the promise shown by these early tests is fulfilled, it means the retirement of the reciprocating engine from ocean service. It is evident that the promoters of the invention have been active, for the Navy Department has now authorized the placing of one of these reduction gears, with a Westinghouse turbine, in U. S. Fleet Collier No. 8, under construction. We are informed that this work is already in hand so that the time requirements of the contractors, the Maryland Steel Co., may be met.

We have received a copy of certain documents submitted to the Navy Department, in presenting the advantages of this combination about to be shown with the all-important speed-reducing gear in an attached casing. It will be seen that all pipe connections are made to the lower half of the turbine casing. High and low-pressure turbines for driving ahead and astern are all included in the same unit and a single pipe brings the high-pressure steam while a single direct exhaust connection for all sections passes to the surface condenser.

The high speed of the turbine rotor, now permissible, allows a reduction in the number of rows of blades and the number is still further reduced by making the first and largest drop in steam pressure through an impulse wheel instead of through a number of rows of reaction blades. It is claimed that, with high pressures and small volumes, the energy is extracted as efficiently as with many rows of the reaction blades. Such a combination of impulse and reaction blades as developed by the Westinghouse Machine Co. for land turbines of large capacity was described in Engineering News, May 28, 1908.

The ahead and astern sections of each unit

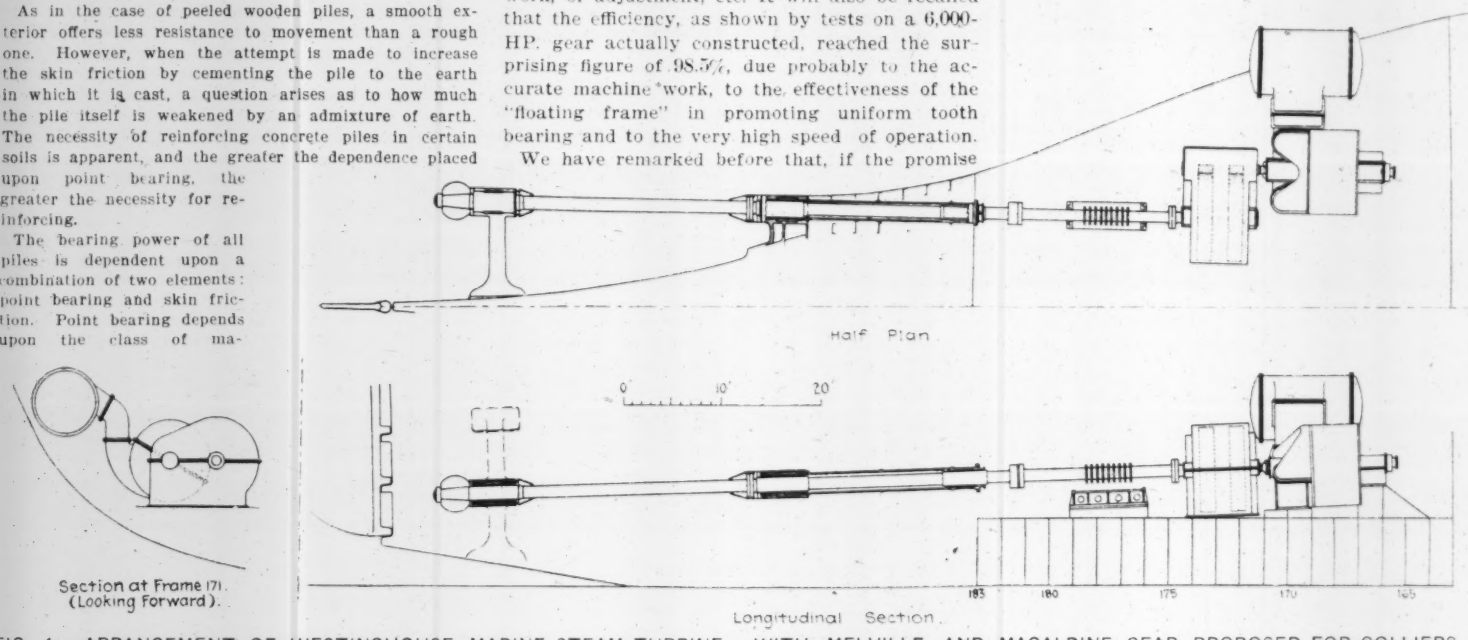

FIG. 1. ARRANGEMENT OF WESTINGHOUSE MARINE STEAM TURBINE, WITH MELVILLE AND MACALPINE GEAR, PROPOSED FOR COLLIERS OF THE "CYCLOPS" TYPE.

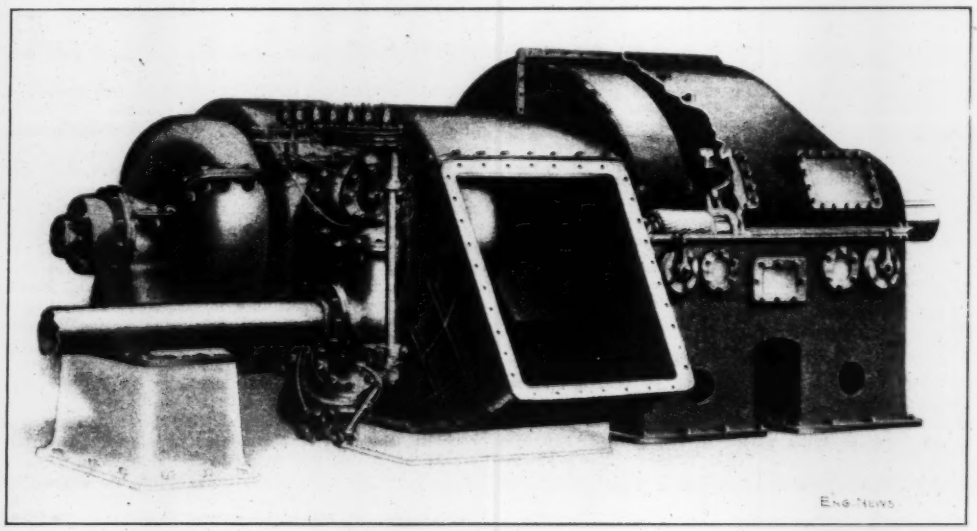

FIG. 2. GENERAL PERSPECTIVE OF WESTINGHOUSE MARINE TURBINE WITH MELVILLE AND MACALPINE GEAR.

installed on Collier No. 8, and we present such matter from them as best shows, in connection with what we have already published, the proposed development. The arrangement laid out for colliers of the "Cyclops" Type is shown in Fig. 1. Fleet Collier No. 8 will have a very similar design. The type of turbine to be used is shown in Fig. 2. A 7,000-HP. unit is there have impulse elements of the same size, though in the latter the amount of reaction blading is reduced. The impulse wheels have two sets of concentric annular passages through the same blades. The outer one, however, is narrower than the inner one. When the turbine is to work at low capacity, one or two nozzles are opened on the outer, or smaller, passage. For higher

*See Engineering News, 1909, July-Dec., pp. 442, 519, 646, 724, 750.

capacities one or two larger nozzles may be opened against the inner, or larger, passages. For maximum loads all nozzles and both sets of passages are used. With a proper proportioning and grouping of nozzles, the range from minimum to maximum throttling can be made without throttling the pressure in the nozzle chambers.

With the reduced number of rows of blades (it is claimed that only 10% of the weight and length of the blading in a direct-connected Parsons marine turbine is necessary) the distance between bearings becomes short and the rotor can be made very stiff and free from vibration.

The cylinder blading, instead of being caulked into grooves in the cylinder, is mounted on flexible bronze strips as shown in Figs. 3 and 4. The cylinder grooves are made wide and deep and are undercut. The strips are slipped into the grooves where these cut through the flanges and light springs hold them up against the shoulders. Steam also is admitted to the space back of the strips, through holes shown, to assist the springs. The bases of the blading strips are extended on one side so that they project over the tops of the corresponding rows of blades on the rotor. The blades are built to have almost no clearance at their ends while in operation, a thing impossible with the rigidly inserted blades. This blading, it is stated, was tried in a 20,000-HP. turbine for 17 months with practically no wear on the blades or the bronze strip.

A notable feature of the proposed installation is the employment of an electro-pneumatic control gear which can be operated from the bridge. Each turbine unit has two independent rotary valves—one for the ahead section and one for the astern. At different stages of rotation the various nozzles of the impulse section are given steam, as already shown possible, to control the capacity of the turbine. These valves are moved by a wing piston worked by air pressure, in a divided cylindrical chamber. By the release of air at proper points the valve travel is stopped as desired. The admission and release of air to the valve motor is controlled by electro-pneumatic valves, but the valve may be moved manually at the turbine if desired at any time.

It is proposed to use, with the condensers for these marine turbines, the Leblanc air pump described in Engineering News, Aug. 19, 1909. It will be remembered that this pump is in effect an ejector connected to the air space of the condenser. On one side of the ejector and just above the throat is a small bladed wheel whose middle plane is in the axis of the ejector. The wheel chamber opens into the bore of the ejector and the wheel projects beyond the opening slightly. Cold water is admitted to the wheel chamber and when the wheel is rapidly revolved a series of slugs of water are thrown down the throat of the ejector at a high velocity. Between each slug, of course, is air and vapor drawn from the condenser. The pump thus operates as a valveless type without clearances and makes possible in the condenser a vacuum which may be as high as 99.7% of that theoretically possible.

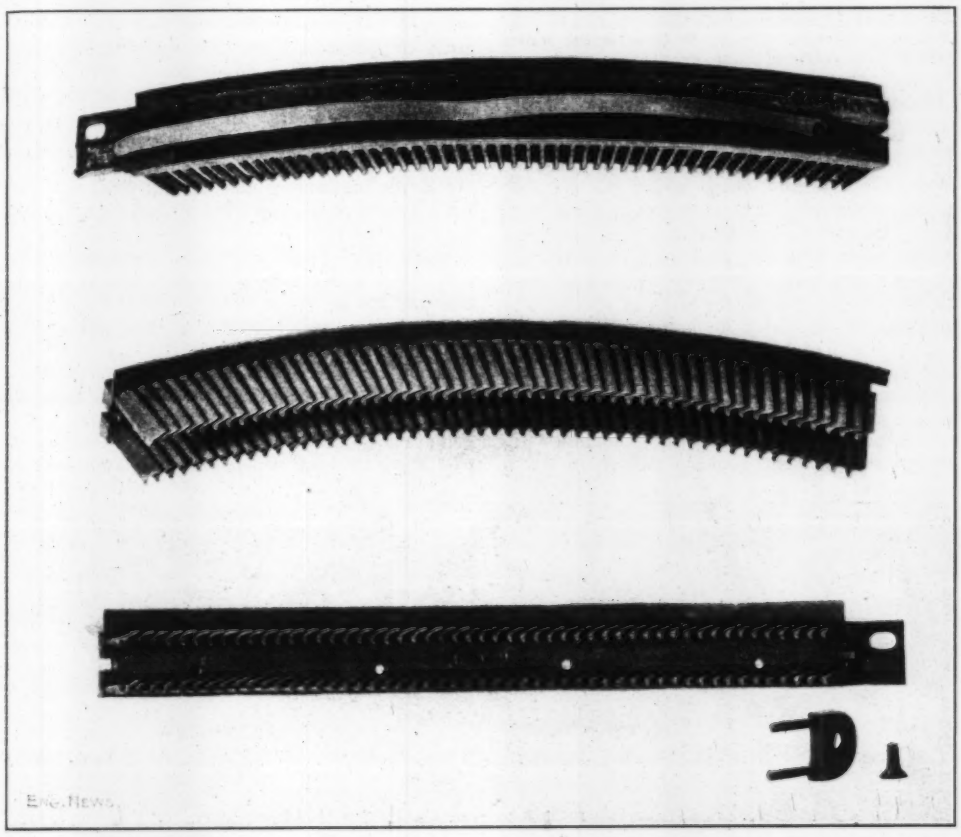

FIG. 3. THREE VIEWS OF ONE SECTION OF SEPARATELY MOUNTED CYLINDER BLADING; WESTINGHOUSE MARINE STEAM TURBINE.

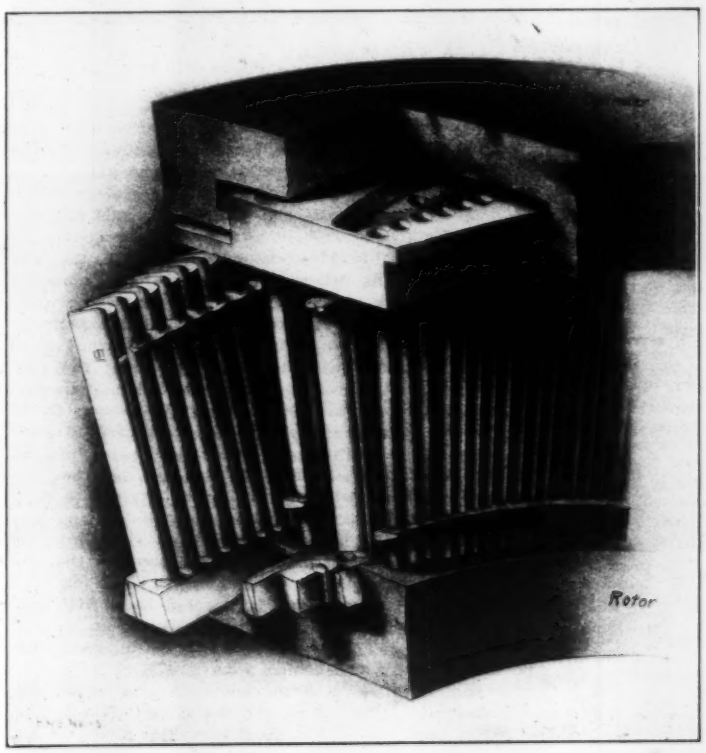

FIG. 4. A SEGMENT OF SEPARATELY MOUNTED BLADING, FOR STEAM-TURBINE STATORS, IN PLACE.

Mallet Pusher Locomotive: Canadian Pacific Railway.

An articulated locomotive of the Mallet duplex type which has been built by the Canadian Pacific Ry. has a number of special features in its design, and was built largely as an experiment with a view to changes in future similar engines for operating on heavy grades. The engine is of the 0-6-6-0 class, and is used as a pusher on the reconstructed line between Field and Stephen, B. C. The maximum grade is 2.2% (formerly 4.4%), and there is practically continuous grade for eleven miles, varying from 1.60 to 2.20%. There are also two spiral tunnels of 573 ft. radius with lengths of 3,200 ft. and 2,890 ft. (Engineering News, Jan. 23, 1908.) The consolidation (2-8-0) engines ordinarily used here and for other heavy grades on this railway have a tonnage rating of 424 tons for summer service on this section, and the Mallet engine has a rating of 660 tons. It handles this load easily, and has handled 700 tons. The two engines compare as follows:

	0–6–6–0	2–8–0
Class	0–6–6–0	2–8–0
Cylinders	23¼ + 34 × 26 ins.	21 × 28 ins.
Driving wheels	4 ft. 10 ins.	4 ft. 10 ins.
Boiler pressure	200 lbs.	200 lbs.
Weight, on drivers	262,000 lbs.	168,000 lbs.
Weight, total	262,000 lbs.	185,000 lbs.
Tractive power	57,400 lbs.	36,200 lbs.
Factor of adhesion	4.57	4.65
Tonnage rating	660 tons	424 tons.

The cylinder arrangement is unusual, the cylinders being placed back to back at the middle of the engine, far enough apart to admit of the removal of the pistons. The high-pressure cylinders are at the forward end of the fixed frames carrying the firebox, while the low-pressure cylinders are at the rear end of the driving truck. This greatly simplifies the arrangement of the piping. The steam from both high-pressure cylinders is delivered to a single receiver or chamber, and this is connected to a similar receiver for the low-pressure cylinders by a 7-in. flexible pipe. In order to give sufficient capacity, this pipe is a vertical inverted U (curved to clear the boiler). One leg is fitted to the end of the high-pressure header, and the connection with the low-pressure header is made directly over the pivot of the truck, so as to minimize the amount of movement.

The low-pressure exhaust pipe is about 11 ft. 9 ins. long, with ball swivel joints at the connection with the low-pressure header and the exhaust nozzle. The maximum swing of the truck on a 20° curve is 5° from the center line of the boiler, and the regular movement of the exhaust pipe is then only 2½°. As the extension of the connections by this movement is only ⅜-in. no packed expansion joint is required. If the low-pressure cylinders had been at the forward end of the truck in the usual way, the angular movement of the exhaust pipe would be over 15°, and the extension of the connection 1⅝ ins. Five packed joints would be required, while the arrangement adopted requires only one packed joint and two ball joints.

The truck frame swivels on a 4-in. pin in an articulated joint carried by cross frames at the rear end of the truck and the forward end of the main frames. This joint is so made with interlocking jaws as to prevent longitudinal rocking of the truck. The center of gravity of the truck is between the rear and middle axles (9 ft. 8 ins. from the hinge), while the boiler is supported at a point between the middle and front axles (14 ft. 10 ins. from the hinge). When the engine is pulling, the hinge pin is in triple shear, but when pushing it is relieved from all stress, the forward casting fitting into a machined socket in the rear casting.

The weight of the boiler upon the truck is 40,000 lbs., being made about 9,000 lbs. greater than that upon the rear group of wheels in order to counteract the tendency to lessen the weight on the truck when pulling or pushing a train. Half of this weight is carried by friction plates and half by a spring-suspended roller, so as to minimize the resistance to the swing of the truck on curves, and the supporting arrangement is such as to bring the boiler and truck in line after passing a curve. Although the guiding is done by this driving truck the engine is found to take curves more easily than the 2-8-0 engines with pony trucks. The flange wear (at contact with the side of the rail head) is also satisfactorily small, in spite of the fact that the line has numerous 10° curves. The wear after about 4,000 miles run was about 3/64-in. on the leading wheels and 1¾-in. on the other wheels.

The steam generating portion of the boiler barrel has tubes only 9 ft. long, but beyond this is a superheater chamber 5 ft. long, followed by a feed-water heater (identical with the steam generating portion) 8 ft. long. A dry pipe from the dome leads the steam to a special dome containing the throttle valve. The superheater consists of 69 vertical double loops of 1¼-in. seamless steel tubes. One end of each loop is connected to a header taking saturated steam from the boiler, while the other end is connected to a header from which the superheated steam is led to the high-pressure cylinders. Two steam jets in the superheater chamber keep the outside surfaces of the tubes free from soot. The top of the chamber is formed by a rectangular casting about 6 ft. 4 ins. wide and 5 ft. long, and this is covered by a steel plate bolted on. Any one loop can be removed or the entire set of loops hoisted out through the top door. In service, the engine uses local Canmore coal having a heat value of about 13,729 B. T. U. This is very fine and dusty, and requires to be thoroughly wetted before firing to prevent loss through the stack.

The engine was designed and built at the railway company's shops in Montreal under the direction of Mr. H. H. Vaughan, M. Am. Soc. M. E., Assistant to the Vice-President (and formerly Superintendent of Motive Power). It was described in considerable detail in the "American Engineer and Railway Journal," from which our information is taken. The general dimensions are given in the accompanying table. As to the cost of operation and maintenance we quote as follows:

> As the locomotive has not been in service sufficiently long, no figures are available as to the cost of maintenance, but it is to be expected that as there is practically double the amount of machinery, this will be somewhat higher than on the consolidation locomotives in the same service.
>
> The operating costs will be slightly higher when considered on a locomotive mile basis. The same crews do the work for the same wages, but more lubricant, waste and sand must of necessity be used, and the cost of wiping and cleaning also will be higher. On a ton-mile basis, which is the fairest comparison for operating costs, it will be lower, due to the greater tonnage hauled, which, it is considered, together with the saving in fuel, will show considerable economy in favor of the Mallet locomotive.

MALLET PUSHER ENGINE: CANADIAN PACIFIC RY.

Class (Mallet duplex compound)	0-6-6-0
Driving wheels (12)	4 ft. 10 ins.
Wheelbase, rigid (two groups)	10 ft. 4 ins.
Total, engine	35 ft. 2 ins.
Engine and tender	60 ft. 7 ins.
Weight (all on drivers)	262,000 lbs.
Weight, engine and tender	391,000 lbs.
Cylinders, h. p. (2)	23¼ × 26 ins.
Cylinders, l. p. (2)	34 × 26 ins.
Valve gear	Walschaerts
Boiler, wagon-top; diameter	5 ft. 0 ins.
Working pressure	200 lbs.
Firebox (radial-stay)	10 ft. × 5 ft. 10 ins.
Tubes; 289; out. diam., 2 ins.; length	9 ft. 1 in.
Tubes (feedwater section), 281; diam., 2 ins.; length	8 ft. 0 in.
Heating surface, tubes	2,605 sq. ft.
Heating surface, firebox	180 sq. ft.
Heating surface, total	2,785 sq. ft.
Superheating surface	420 sq. ft.
Equivalent heating surface (heating surface + superh. surface × 1.5)	3,415 sq. ft.
Grate area	58 sq. ft.
Coal in tender	12 tons
Water in tender	6,000 U. S. gals.

A Motor Wagon for Transporting Structural Material in City Streets.

The transportation of long and heavy beams, girders, columns and other structural material through city streets is often a troublesome matter, and ordinarily the difficulties are increased by the long teams of horses necessary to handle the load. An interesting machine which has been built recently in Chicago for work of this kind is a powerful steam-operated motor wagon, which occupies less length of the street and is more easily handled than a team of six or eight (or more) horses.

This machine is shown in the accompanying cut transporting a very long steel column. Its carrying arrangement consists essentially of a pair of 10 × 12-in. Douglas fir beams or sills supported at a considerable height above the ground by bolsters resting upon the axles. The front bolster rests upon the fifth wheel on which the front axle swivels. Across the sills are two shafts (with ratchets and levers) for chains by which the load is suspended beneath the sill. The steering wheel and seat are also mounted on the sills. The machine is mounted on wheels of very substantial construction, having heavy ¾-in. steel tires. The front wheels are 4 ft. 6 ins. diameter with tires 12 ins. wide; the rear (and driving) wheels are 6 ft. 8 ins. diameter, with tires 18 ins. wide. The wheelbase is 26 ft. 6 ins. The rear axle is arched upward to clear the load. With a very long piece, extending far beyond the machine, the rear end may be supported by a small wheeled truck chained to it.

Over the rear axle is mounted the driving machinery, which is enclosed in a steel cab. A vertical inverted marine engine supplies the motive power; this has two cylinders 8 × 8 ins., and runs at 225 r. p. m. It is rated at 30 HP., but can develop 40 HP. when working at its full capacity. The engine drives a 10-in. phosphor-bronze pinion meshing with a 37½-in. cast-steel compensating gear on a 3⅞-in. shaft; the differential gear is of the usual bevel-gear type. On each end of this shaft is a 9-in. pinion (4 ins. face) meshing with an internal gear riveted to the inside of the driving wheel tire The phosphor-bronze main pinion and the cast-steel compensating gear are 5 ins. wide on the face and have cut teeth, so that the gearing works with very little noise. The gear ratio is 1 to 55, or 55 revolutions of the engine to one revolution of the driving wheels.

Sufficient clearance and backlash are provided between the driving pinions and the internal gears to permit of the wagon operating with one driving wheel 12 ins. lower than the other (as in the event of one wheel running in a rut) without binding the gears. In case the wagon should become mired or stalled the differential gear can be locked immediately; ample power being provided, the machine is able to pull itself out of a rut 12 to 15 ins. deep even if both rear wheels become stalled.

Steam is supplied by a marine water-tube boiler 5 × 5 ft., with a height of 4 ft. 8 ins.; this has 330 sq. ft. of heating surface and 13 ft. of grate area, and carries a working pressure of 130 lbs. The fuel is a specially selected combination of hard and soft coal, which gives no smoke and shows only a brown vapor. The exhaust steam is passed through a muffler before going to the stack. The machine carries about half a ton of coal, which is almost sufficient for a day's work; it has also a water tank with a capacity of 260 gals., or sufficient for two miles run.

A STEAM MOTOR WAGON (30 HP.) FOR TRANSPORTING LONG AND HEAVY STRUCTURAL MATERIAL IN CITY STREETS.

Geo. W. Jackson (Incorporated), Chicago, Builder and Owner.

The length of the machine over all is 32 ft.; and it has a height of 6 ft. 10 ins. to the top of the sills and 11 ft. 6 ins. to the top of the cab. Its width over all is 8 ft. 6 ins. The light weight is about 12 tons, and the load carrying capacity is about 20 tons. The maximum theoretical speed, when running light, is 8,800 ft. per hour, but the average speed in service is about 1¼ to 1½ miles per hour.

The machine was proposed by Mr. Geo. W. Jackson, President and Engineer of Geo. W. Jackson (Incorporated), Chicago, for use in handling structural material, etc., for that firm's contracts on engineering construction work. It was designed and built by the machinery department of the same firm.

Core Drilling Under the Hudson River for the Catskill Aqueduct.

By WILLIAM E. SWIFT,* Assoc. M. Am. Soc. C. E.

Continuous diamond drill holes have now been driven from each shore to the middle of the Hudson River, on the line of the deep tunnel crossing for the Catskill aqueduct. Each drill has been in solid granite from start to finish, thus

*Division Engineer, Hudson River Division, Board of Water Supply of the City of New York, Cornwall-on-Hudson, N. Y.

proving beyond peradventure that at their depth the rock is uninterrupted from side to side. This granite is the same as that exposed in Storm King Mountain on the west bank and Breakneck on the east. From a quarry in Breakneck was taken the material from the well-known High Bridge which carries the old Croton aqueduct over the Harlem River in the upper part of New York City, built in 1848. The test shafts from which this drilling has been done have been deepened while the drill work was in progress. The East Test Shaft has now attained a depth of 755 ft. and the West Test Shaft is 695 ft. deep. By these means, the character of the rock and the water to be anticipated in tunneling are being developed with great thoroughness.

In Engineering News, April 2, 1908, p. 358, there was printed an article describing the general drill explorations which led to the selection of this site about four miles above West Point for the Hudson River crossing of the aqueduct. This article described in some detail the methods of vertical drilling from scows anchored in the river, the beginnings of the test shafts and the difficulties encountered; it also reviewed the geological studies of the region. During the two years that have elapsed, difficulties and troubles have continued to pursue the river drilling. Nevertheless, a persistent endeavor has been concentrated on one hole exactly in the middle of the river. The work has been done by the Phoenix Construction Co. under the engineering direction of their Mr. F. W. Belknap, with Mr. George A. Hammond as resident superintendent.

The hole in the middle of the river was started with an 18-in. casing which was carried to a depth of about 250 ft. below the river surface. Inside the 18-in. casing, there were telescoped successively, 14-in., 10-in., 8-in., 6-in. and 4-in. casings, each size being forced as far as possible without injuring the pipe. In telescoping in hard material, each new line of pipe could be driven only about 100 ft. below the bottom of the next larger casing. A 4-in. wash pipe was used in sinking the casing, the coarser material including some cobbles 4 to 6 ins. in diameter which were bailed out with a sand bucket. These buckets, of the oil well type, were of different diameters to 10 ins., made of a length of casing from 6 to 8 ft. long fitted with a clapper valve at the bottom, the bail being attached to a piston which sucked the material into the bucket. Small boulders and hard material were passed by blasting or by chopping with an X-bit attached to the wash pipe. Large boulders were drilled with a diamond drill and then shattered with dynamite so as to allow the casing to pass. Several boulders from 1 to 6 ft. in diameter were encountered in this hole. At a depth of about 693 ft. below river surface, material was encountered that required drilling and some granite core was recovered. This material was penetrated for about 15 ft. when floating ice forced the discontinuance of work for the season. At times rapid progress was made (20 to 25 ft. per day) but owing to numerous obstructions encountered, eight months were consumed in sinking 700 ft. The casings were all disconnected below the river surface and work is soon to be resumed by reconnecting the casings.

In February, 1907, an agreement was made for sinking a test shaft on each shore of the river and drilling inclined diamond drill holes from these shafts. Early in December of the same year, the contractor abandoned this work when the East Shaft was 280 ft. deep below the original surface and the West Shaft 265 ft. No drilling was done under this agreement. After losing nearly a year with legal delays, work was resumed by the Board with its own forces. At a depth of approximately 250 ft. in each shaft, large chambers to accommodate the diamond drills were excavated on the river side and shaft sinking continued. After sinking the shafts 40 to 50 ft. below the drill chambers, the diamond drills were installed and shaft sinking continued. Thereafter, shaft sinking and diamond drilling progressed simultaneously.

The rock in both shafts is very hard and tight, requiring about 40 drill holes in each round (the shaft being circular and about 17 ft. in diameter). With drill holes 9 ft. deep, the rock is shattered to a depth of 7.5 to 8 ft. A progress of 50 to 60 ft. per month of timbered shaft is now being made.

Large chambers were excavated about 400 ft. below the river surface for high-pressure pumps of the outside-packed, plunger pattern. These relay pumping stations are to be about 400 ft. apart and have a considerable reservoir capacity. An occasional concrete ring is put in back of the shaft timbering and small pumps installed to lift the water so collected to the station pumps. In the East Shaft, the leakage is about 175 gals. per min. while at the West Shaft it is less than 40 gals. per min.

Compressed air is used except on the mine hoists, which are operated by steam, and the station pumps, which are also run on steam, the exhaust being led to a suction condenser close to the pump on the suction pipe.

For the drilling beneath the river from the chambers in the shafts, an agreement was made April 20, 1909, with Messrs. Sprague & Henwood, of Scranton, Pa., who began work in May, 1909, with Mr. N. R. Osborne as superintendent. The first two holes have been completed. The hole from the East Shaft attained a length of 1,834 ft. and its extremity is about 1,480 ft. below the river surface. The hole from the West Shaft just completed is 2,051 ft. long and its end about 1,500 ft. below the river surface.

Diamond drills capable of drilling 3,000 ft. were installed and operated by compressed air. A hole 3½ ins. in diameter was drilled a few feet in the rock, and in it a 3-in. pipe was grouted. This pipe projected a short distance and was equipped with a gate valve for checking any flow of water. The holes were started 2⅝ ins. in diameter and two reductions made to 2 1-16 ins. and 1½ ins. in diameter, the last size giving a core ⅞-in. in diameter. The rods were pulled in 30-ft. lengths and at the maximum depth the time of pulling and replacing the rods was about five hours. The average progress per day of three eight-hour shifts was about 12 ft., but about eight months were required for drilling 2,050 ft., including delays and no Sunday work. The maximum flow of water from the drill hole in the West Shaft was about 5 gals. per min., but at a depth of about 700 ft. in the east drill hole, 200 gals. per min. came from the 2⅝-in. hole. This large flow of water was reduced to 5 gals. per min. by inserting flush joint casing and reducing the diameter of the hole. The flow later increased to about 90 gals. per min. This water contained 4,500 parts of chlorine and 1,200 parts of calcium per million, while the river water at the same time ran only 2,000 parts of chlorine and 90 parts of calcium per million. Continued tests of the drill hole water proved that it did not decrease in salinity and was evidently fossil water and had no direct connection with the river.

In the drill hole from the East Shaft, the upward current of water was so great at times that three men could with greatest difficulty force the rods down into the hole (although the angle was about 43° from the horizontal) and at times the drill engine was used to force the rods back into the hole. This upward pressure made it difficult to secure the necessary circulation of water, at the bit, pumped down through the drill rods to wash up the grinding and as a result two bits became clogged and burned fast. One was recovered by reaming and one is still fast in the bottom of the hole. In the one recovered, the heat was sufficient to fuse the metal in the bit and the granite core. Some of the diamonds were found buried in the fused material that was once granite core.

The east hole was started at an angle of 43° from the horizontal and the west hole at an angle of 38° from the horizontal. Both showed a tendency to dip downward for the first few hundred feet but later came up slightly. This was due to some extent to the use of a nickel steel core barrel, for the first 600 or 700 ft., of almost the same diameter as the bit with four small diamonds set in the outside of the barrel about 4 ins. above the bit to keep up the gage. Beyond this depth a core barrel tapered about 3 16-in. was used, with the large end next to the bit, to make the hole curve upwards. Attempts to make the hole curve upwards were not entirely satisfactory.

The dip of the holes was determined every 50 or 100 ft. by means of a solution of hydrofluoric acid in a glass vial. Special couplings were bored to contain these tubes so that their long axes were parallel with the center line of the rods. The vials were lowered and allowed to remain at rest about 20 minutes or until a clear line was etched on the glass at the surface of the acid solution. Two or more tubes were lowered at one time, usually at 50-ft. intervals along the rods. After removal from the hole, the angle made by the etched line with center line of the vial was read and a negative correction for capillarity and refraction applied. This correction is a maximum at an angle of 45° amounting to 7° or 8° and was determined by experiment. Duplicate observations proved that with a vial ⅞-in. in diameter the readings checked within 30".

Experiments are being made with a pressure gage designed by Messrs. Kalmus & Lewis, of Massachusetts Institute of Technology, to check the observations taken with the hydrofluoric acid apparatus. This mercurial gage is ⅞-in. diameter and about 3 ft. long and registers maximum pressures.

The Hudson River crossing has attracted much attention but it is only a connecting link in the great system of aqueducts and reservoirs for bringing to New York City an additional supply of water from the Catskill Mountains, which is being successfully constructed by the Board of Water Supply, of which Mr. John A. Bensel, President, Am. Soc. C. E., is President, and Mr. Charles N. Chadwick and Mr. Charles A. Shaw are Commissioners; Mr. J. Waldo Smith, M. Am. Soc. C. E., is Chief Engineer, Mr. Charles L. Harrison, M. Am. Soc. C. E., until March 31, 1910, Deputy Chief Engineer, and Mr. Robert Ridgway, Engineer of the Northern Aqueduct Department; all of whom have given much personal attention to the explorations of the Hudson River crossing.

THREE AERONAUTS WERE KILLED at Stettin, Germany, on April 3, during an ascent of the balloon "Pommern," which has made several flights in this country. The ascent was attempted during a gale, it is reported, and the balloon broke away with its four passengers before preparations were complete. First it swung into telegraph wires on rising and smashed a factory stack, injuring the occupants of the car. After traveling some 36 miles at an altitude of 6,000 ft., it was blown out over the Baltic Sea and fell. A tug, which was nearby, rescued two of the men, but one soon died from his injuries.

Hubert Le Blon, a well-known French automobilist, a former member of the Balloon Corps of the French army and a consulting engineer, was killed at San Sebastian, Spain, on April 2, during an aeroplane flight. His motor failed and he was unable to adjust his guide planes for an easy glide to earth. The type of aeroplane has not been reported.

Richard Abegg, professor of chemistry at Breslau, Prussia, was killed at Breslau on April 4, while making a landing after a balloon flight. The trip had been made in strong winds and the basket was dragged over the ground after the two other occupants had alighted.

THE AUSTIN DAM, across the Colorado River at Austin, Tex., is to be rebuilt by the Dumont-Holmes Concrete Co., of Detroit, Mich., under a contract with the city of Austin under which the city pays for the dam in 40 semi-annual instalments of $25,000 each. At a special election held at Austin April 5, the contract for rebuilding the dam was approved by a vote of 2,187 to 38. The dam will create a head of about 60 ft., and the power will be used to operate the municipal water and lighting plant, while additional power will be developed for sale.

A COST-PLUS A FIXED SUM CONTRACT with a notable variation has been executed for the construction of an irrigation system in Colorado. The contract is between the Costilla Estates Development Co., of Colorado Springs, and the Wogan-Wilcoxen Construction Co., of Denver. The basis of the contract is actual cost plus an agreed sum for profit; but the company will also pay the contractors, as an incentive to economy, one-third of the amount saved under an agreed fixed maximum price, and the contractors agree to share one-third of the amount by which the fixed maximum price is ex-